

Technical Report

Wave Run-up and Wave Overtopping at Dikes

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Technical Report Wave Run-up and Wave Overtopping at Dikes



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Cover photograph

Road & Hydraulic Engineering Institute

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Note to this English version

This report is from Dutch origin and is a translation into English. In the Netherlands it is used as a guideline for safety assessment and design of dikes. Assessment of the required dike heights for wave run-up and wave overtopping is important in the Netherlands and has a long history. Some parts of this report, therefore, refer to typical Dutch situations.

Nevertheless, the methods given in the report to determine wave run-up and wave overtopping are for general applications.

This Technical Report entitled *Wave run-up and wave overtopping at dikes* has been composed under the auspices of the TAW and has been based on an investigation [WL, 1993-1] *Wave run-up and wave overtopping at dikes*, which has been supplemented with additional research and recent views on some less developed aspects.

Up to the first half of the 1990s, the *Guidelines for the Design of River Dikes, part 2* [TAW, 1989] were mainly consulted for determination of wave run-up and wave overtopping. In Appendix 11 of these guidelines, formulae are presented for wave run-up and wave overtopping, most of which were published earlier in the TAW report *Wave run-up and wave overtopping* [TAW, 1972].

Considering that wave run-up heights and wave overtopping discharges are greatly involved in the determination of the total crest height of a dike, it is more than obvious that a great deal of study has been carried out in recent years into these aspects. As a result, a large amount of knowledge has been acquired over time in the area of the influence of roughness, slope angle, berms, angle of wave attack and vertical walls on wave run-up and wave overtopping. Results on the effects of shallow and very shallow foreshores have also been received recently.

Although the formulae for determining wave run-up and wave overtopping were until recently intended for deterministic calculations, they are now regularly being applied in probabilistic calculations, in which the distribution of the input data and uncertainty in the constants are included. This puts strict requirements on the formulae with regard to the continuity and validity of the functions.

A great deal of experience has already been gained by various users from the intermediate results of the study and draft versions of this report. Recommendations from the users have led to improvements in the usefulness of the new formulae. The areas of validity of the new formulae have also been determined. This does not mean that the formulae can be applied to every profile and all wave conditions without exception. Indeed, it is for these complex situations outside the areas of validity that craftsmanship will still be required.

The new wave run-up and wave overtopping formulae replace the existing formulae as given in the *Guideline for design of river dikes, part 2* [TAW, 1989]. The new formulae can be applied in the design and safety assessment procedures for river dikes. The new formulae will also be or even are being included in the safety assessment procedure for the dikes along the IJsselmeer.

For dikes along the coast and estuaries, sometimes shallow or very shallow foreshores occur which lead to deviant wave spectra, possibly in combination with long waves. Although research has not yet completely crystallised, it has been decided to include recent results and to adjust the formulae where necessary so that they can also be applied in this type of situation. Specially, for very shallow foreshores the wave run-up turns out to be a little higher than in the past.

Although there is a considerable body of knowledge relating to dimensioning based on wave-run-up and wave overtopping, it has not yet been fully developed with regard to the following aspects:

- determination of representative wave boundary conditions at very shallow areas;
- guidelines for required strength, particularly under oblique wave attack and wave overtopping;
- wave transmission at oblique wave attack and;
- wave growth under extreme winds.

Research on these items will, as a further development of this Technical Report, be initiated, as wave run-up and wave overtopping have considerable influence on the determination of required dike heights.

This Technical Report is part of a series of Technical Reports and Guidelines as mentioned in the *Fundamentals on Water Defences* [TAW, 1998-1]. This means that all formulae on wave run-up and wave overtopping, which have been published before by the TAW, dispose of now.

The Hague, May 2002

W. van der Kleij

Chairman, Technical Advisory Committee on Flood Defence

1.1 Background to this report

In 1993, a report appeared with the same title as the current report [WL, 1993-1] and in 1997, a revised version appeared [WL, 1997-1]. Draft versions of the technical report were converted into the TAW framework with some last amendments based on experience with the accompanying program PC-OVERSLAG. In the last round of editing, the influence of shallow and very shallow foreshores was quantified, which has led to some adjustment to the formulae and other wave parameters.

The 1993 report is a summary of the (new) study results that were then available concerning wave run-up and wave overtopping for dikes. This summary was intended to make the study results easier to use when designing and evaluating dikes. Although we have attempted to make all the formulae as broadly applicable as possible with regard to their application, after several years of intensive practical use it appears that practical situations are almost never exactly the same as those in the schematisations by which the study was performed. For example, situations often occurred with more than one slope in a single dike profile, and sometimes even combined with more than one berm. The areas of application of the new formulae are now indicated, together with the possibilities for interpolation in other situations.

Background information on the study, on which the 1993 report was based, can be found in the extensive study report by Van der Meer and De Waal [WL, 1993-2]. The study into desired amendments to the 1993 report was also published [WL, 1997-2].

In brief, the changes that were brought in relation to the first report from 1993 are explained below:

- The definitions in the application area have been more accurately formulated. This concerns mainly slopes, berms, foreshores and wave run-up and wave overtopping themselves. The definitions are brought together in paragraph 1.2. For situations that are not covered by the definitions (a slope that is too flat or a berm that is too steep or too long) estimates of wave run-up and wave overtopping can be made by interpolation.
- The wave height that is used in the calculations is the significant wave height at the toe of the dike.
- Determination of an average slope and the description of the influence of a berm were simplified and accentuated as was that for average roughness.
- The formulae were made continuous where necessary and if possible they were also simplified, especially for:
 - The influence factor for the position of height of the berm;
 - Wave overtopping in the transition zone between breaking and non-breaking waves;
 - The influence factor for the angle of wave attack for very large wave angles.
- The influence factor for a shallow foreshore has been removed.
- The influence of wave overtopping of a vertical wall on a slope can be described by the influence factor.

After publication of the amended report [WL, 1997-1], further study was carried out into one aspect that had not been intensively studied before: the effect of shallow and very shallow foreshores and the breaking of waves on wave run-up and wave overtopping. These results were published in the study report [WL, 1999-2]. Although the study has not provided sufficient explanations for all effects, it was decided to integrate the results as much as possible into the current report. This has led to the following changes in comparison to the 1997 version:

- For the significant wave height at the toe of the structure, the spectral measure H_{m0} has been used.
- For the representative wave period, the peak period is no longer used, but the spectral peri-

od $T_{m-1,0}$. For 'normal' spectra with a clear peak, $T_{m-1,0}$ lies close to the peak period T_p and a conversion factor is given for a case for which only the peak period is known.

- Using the above-mentioned spectral period, it is no longer necessary to have a procedure for double-peaked or bi-modal spectra, and this procedure has been removed.
- Formulae for wave run-up and wave overtopping have been adjusted to the use of the above mentioned parameters, specifically:
 - The maximum for wave run-up lies higher than in the previous versions and progresses more fluidly from breaking to non-breaking waves.
 - The formulae for wave overtopping have only been adjusted to use of the above mentioned parameters. For shallow and very shallow foreshores separate formulae are given.

These last changes have been justified in a background report [DWW, 2001].

1.2 Definitions

In the list of symbols short definitions of the parameters used have been included. Some definitions are so important that they are explained separately in this section. The definitions and validity limits are specifically concerned with application of the given formulae. In this way, a slope of 1:12 is not a slope and it is not a berm. In such a situation, wave run-up and wave overtopping can only be calculated by interpolation. For example, for a slope of 1:12, interpolation can be made between a slope of 1:8 (mildest slope) and a 1:15 berm (steepest berm).

Foreshore

A foreshore is a part in front of the dike and attached to the dike, and can be horizontal or up to a maximum slope of 1:10. The foreshore can be deep, shallow or very shallow. In the last case, the limits of depth mean that a wave can break on this foreshore and the wave height is therefore reduced. The wave height that is always used in wave run-up and wave overtopping calculations is the incident wave height that should be expected at the end of the foreshore (and thus at the toe of the dike).

Sometimes a foreshore lies very shallow and is rather short. In order for a foreshore to fall under this definition, it must have a minimum length of one wavelength L_0 . After one wavelength, the wave height would be reasonably adjusted to the shallow or very shallow foreshore and the wave height at the end of this foreshore can be used in the formulae. If the shallow or very shallow foreshore is shorter, then interpolation must be made between a berm of $B = 0.25.L_0$ and a foreshore with a length of $1.0.L_0$. In the Guidelines [TAW, 1989], a minimum length of 2 wavelengths was used and it was suggested that, for a shorter length than one wavelength, no reduction for wave height would be applied and the foreshore would be ignored. Current insight suggests rather that most waves will break on a shallow or very shallow foreshore within one wavelength and that this wavelength can be used as the lower limit.

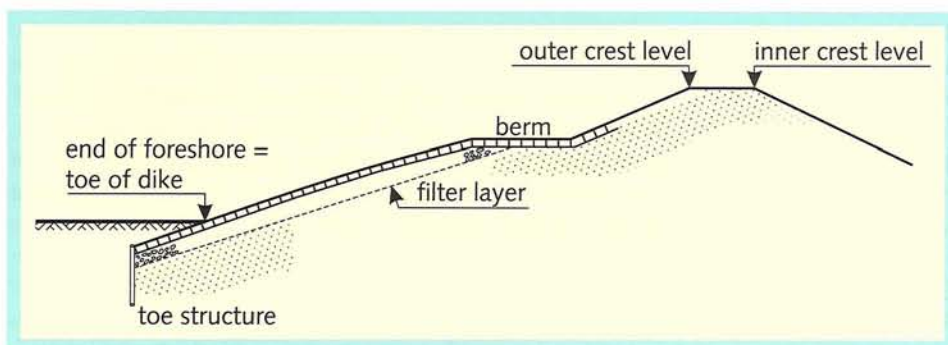
A precise transition from a shallow to a very shallow foreshore is hard to give. At a shallow foreshore waves break and the wave height decreases, but still a wave spectrum exists with more or less the shape of the incident wave spectrum. At very shallow foreshores the spectral shape changes drastically and hardly any peak can be detected (flat spectrum), as the waves become very small due to breaking and many different wave periods arise. Generally speaking the transition between shallow and very shallow foreshores can be indicated as the situation where the original incident wave height, due to breaking, has been decreased by 50% or more. The wave height at a structure on a very shallow foreshore is much smaller than in deep-

water situations. This means that the wave steepness, as defined in this report, becomes much smaller too. Consequently, the breaker parameter, which is used in the formulae for wave run-up and wave overtopping, becomes much larger. Values of 4 – 10 for the breaker parameter are possible then, where maximum values for a dike of 1:3 or 1:4 are normally smaller than 2 or 3. Another possible way to look at the transition from shallow to very shallow foreshores, is to consider the breaker parameter. If the value of this parameter exceeds 5-7, then a very shallow foreshore is present (unless a very steep slope is present, much steeper than 1:3). In this way no knowledge about wave heights at deeper water is required to distinguish between shallow and very shallow foreshores.

Toe of dike

In most cases, it is clear where the toe of the dike lies, which is where the slope changes into the foreshore. It is actually possible that this foreshore has a changing bottom, such as for example a tideway in front of the dike. In such a case the position of the toe is not constant. During design of a dike, we have to estimate where the foreshore lies or will lie under the design conditions and this also determines the position of the toe of the dike. This same situation applies for a safety assessment of a dike. For measuring wave run-up, the foreshore profile available at that moment must be used for verification, and the wave height at the position of the toe of the dike.

Figure 1:
cross-section of a dike
showing the outer slope



Wave height

The wave height used in the wave run-up and wave overtopping formulae is the incident significant wave height H_{m0} at the toe of the dike, called the spectral wave height, $H_{m0} = 4 \sqrt{m_0}$. Another definition of significant wave height is the average of the highest one third of the waves, $H_{1/3}$. This wave height is thus not used. In deep water, both definitions produce almost the same value, but situations in shallow water can lead to differences of 10-15%.

In many cases a foreshore is present on which waves can break and by which the significant wave height is reduced. In the Guidelines [TAW, 1989], a simple method for determining depth-limited wave heights is given. There are models that in a relatively simple way can predict the reduction in energy from breaking of waves and thereby the accompanying wave height at the toe of the structure. The wave height must be calculated over the total spectrum including any long-wave energy present.

Based on the spectral significant wave height, it is fairly simple to calculate a wave height distribution and accompanying significant wave height $H_{1/3}$ using the method of Battjes and Groenendijk [BG, 2000].

Wave period

The wave period used for wave run-up and wave overtopping is the spectral period $T_{m-1.0}$ (m_{-1}/m_0). This period gives more weight to the longer period in the spectrum than an average period and, independent of the type of spectrum, gives the corresponding wave run-up or wave overtopping for the same values and the same wave heights. In this way, wave run-up and wave overtopping can be easily determined for double-peaked and 'flattened' spectra, without the need for other difficult procedures.

In the case of a uniform spectrum with a clear peak there is a fixed relationship between the spectral period $T_{m-1.0}$ and the peak period. In this report a conversion factor ($T_p = 1.1 \cdot T_{m-1.0}$) is given for the case where the peak period is known or has been determined, but not the spectral period.

Slope

Part of a dike profile is a slope if the slope of that part lies between 1:1 and 1:8. These limits are also valid for an average slope, which is the slope that occurs when a line is drawn between $-1.5 H_{m0}$ and $+z_{2\%}$ in relation to the still water line and berms are not included (see figure 7 and section 2.3). A continuous slope with a slope between 1:8 and 1:10 can be calculated in the first instance using the formulae, but the reliability is less than for steeper slopes.

Berm

A berm is part of a dike profile in which the slope varies between horizontal and 1:15. The position of the berm in relation to the still water line is determined by the depth d_h , the vertical distance between the middle of the berm and the still water line. The width of a berm, B , may not be greater than one-quarter of a wave length, i.e., $B < 0.25 \cdot L_0$. If the width is greater, then the structure is between that of a berm and a foreshore, and wave run-up and wave overtopping can be calculated by interpolation.

Crest height

The crest of a dike, especially if a road runs along it, is in many cases not completely horizontal, but slightly rounded and of a certain width. In the Guidelines for the Design of River Dikes [TAW, 1985] and [TAW, 1989] the crest height is not precisely defined. In the Guideline on Safety Assessment [TAW, 1999-1] crest height is defined as the height of the outer crest line. This definition therefore is used for wave run-up and wave overtopping. In principle the width of the crest and the height of the middle of the crest have no influence on calculations for wave overtopping. Of course the width of the crest, if it is very wide, can have an influence on the allowable wave overtopping.

The crest height that must be taken into account during calculations for wave overtopping for an upper slope with quarry stone is not the upper side of the quarry stone. The quarry stone layer is itself completely water permeable, so that the under side must rather be used. In fact the height of a non- or only slightly water-permeable layer determines the crest height in this case for calculations of wave overtopping.

Wave run-up height

The wave run-up height is given by $z_{2\%}$. This is the wave run-up level, measured vertically from the still water line, which is exceeded by 2% of the number of incoming waves. The number of waves exceeding this level is hereby related to the number of incoming waves and not to the number that run-up.

A very thin water layer in a run-up tongue cannot be measured accurately. In model studies the limit is often reached at a water layer thickness of 2 mm. In practice this means a layer thickness of about 2 cm, depending on the scale in relation to the model study. Very thin layers on a smooth slope can be blown a long way up the slope by a strong wind, a condition that cannot be simulated in a small-scale model too. Running-up water tongues less than 2 cm thick actually contain very little water. Therefore it is suggested that the wave run-up level is determined by the level at which the water tongue becomes less than 2 cm thick. Thin layers blown onto the slope are not seen as wave run-up.

Wave overtopping

Wave overtopping is the average discharge per linear metre of width, q , for example in m^3/s per m or in l/s per m. Wave overtopping is calculated in relation to the height of the outer crest line and it is assumed that this wave overtopping also reaches the rear of the slope and the inner slope.

In reality there is no constant discharge over the crest of a water defence during wave overtopping. The highest waves will push a large amount of water over the crest in a short period of time, less than a wave period. Lower waves will not produce any wave overtopping. In this report a method is given by which the distribution of wave overtopping volumes can be calculated for each wave. Such a wave overtopping volume per wave, V , is given in m^3 per m per wave.

1.3 Determination of wave height and wave period at toe of dike

In Chapter 5 of the Guidelines [TAW, 1989] it is shown how wave conditions can be determined. In addition of course there are more advanced computer models that enable determination of the wave conditions close to the dike. It is recommended that the most accurate method possible should be selected. The method used most at this time is the program SWAN. This program provides wave heights not very different from actual measured values even for shallow and very shallow foreshores. The program does not provide reliable wave periods in this case, as explained in the following section.

For safety assessment of water defences, wave conditions are given in the Hydraulic Boundary Conditions 2001, HR2001 [RWS, 2001]. No distinction is made between H_{m0} or $H_{1/3}$ in this book and no values are given for the spectral period $T_{m-1,0}$.

The hydraulic boundary conditions mentioned above are given at a certain location. Very often this is 50 m – 200 m from the toe of the dike. For calculation of wave run-up or wave overtopping the wave height at the toe of the dike has to be determined. If depths at the given location and the toe of the structure are similar, than the given values can be used. If a sloping foreshore is present it can be required to calculate the wave height at the toe of the dike. If a very shallow foreshore is present between a given location and the dike, it is suggested to consult a specialist.

The spectral period is a new parameter in the area of wave conditions for safety assessment and design of water defences. In the future it is expected that this period will be included in new versions of the Hydraulic Boundary Conditions. As long as it is not included, conversion of the given periods must be made and in specific cases, such as very shallow foreshores, the spectral period must be determined separately.

Calculation of the spectral period $T_{m-1,0}$ on the basis of measured or calculated spectra is a very simple task. It is still possible for very shallow foreshores to calculate the correct spectral type and thereby the correct spectral wave period. Only Boussinesq-models appear to be capable of this and they are mainly used by specialists. Determination of the correct wave period for heavy and very heavy breaking waves on a shallow foreshore will still require specialised experts for the time being.

1.4 General calculation procedure for wave run-up and wave overtopping at a simple slope

In chapter 2 a general formula for wave run-up will be given, including all kinds of influence factors for example for a berm, roughness on the slope and oblique wave attack. Chapter 3 gives the formulae for wave overtopping. As a dike profile can be very complex (more slopes and/or berms, different roughness per slope section), the program PC-OVERSLAG has been developed.

In this section an overall view is given in which order various parameters have to be calculated and where to find the formulae. The procedure is valid for a simple slope with roughness, a berm and oblique wave attack at relatively deep water (not much wave breaking).

Calculation procedure	Chapter	Formula
1. Determine wave conditions at toe of dike: H_{m0} , $T_{m-1,0}$	1.3	
2. Calculate influence factor for angle of wave attack γ_β	2.5	8
3. Adjust wave conditions if $\beta > 80^\circ$	2.5	
4. Calculate average slope, $\tan \alpha$	2.3	Fig. 7
5. Calculate $z_{2\%, \text{smooth}}$ (smooth: for $\gamma_b = 1$ and $\gamma_f = 1$)	2.2	3
6. Calculate influence factor for roughness on slope γ_f	2.7	19, 20 and appendix 1
7. Calculate $z_{2\%, \text{rough}}$ (rough: for $\gamma_b = 1$)	2.2	3
8. Calculate influence factor for berms γ_b	2.6	13
9. Calculate 2% wave run-up	2.2	3
10. Calculate γ_β for wave overtopping	2.5	9
11. Calculate wave overtopping with above γ_b and γ_f	3.1	22 and 23
12. Calculate overtopping volumes per wave	3.4	28-32

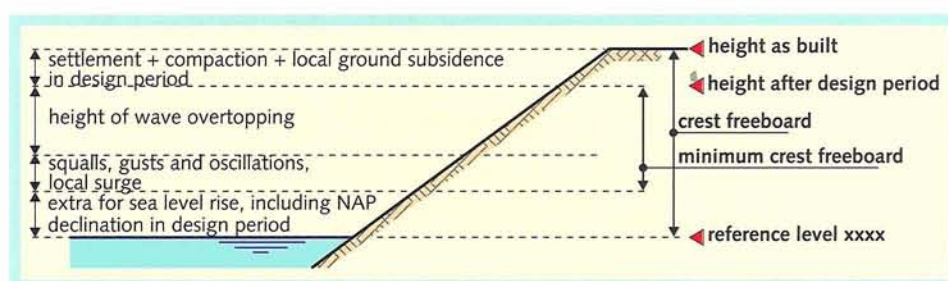
2. Wave run-up

2.1 General

Dikes in the Netherlands have a rather gently sloping outer slope, usually less than 1:2. A dike consists of a toe structure, an outer slope often with a berm, a crest of a certain width and outer and inner crest lines, and an inner slope (see figure 1).

During design or safety assessment of a dike, the crest height does not just depend on wave run-up or wave overtopping. Account must also be taken of a reference level, local sudden gusts and oscillations (leading to a corrected water level), setting and an increase of the water level due to sea level rise.

Figure 2:
important aspects during calculation or assessment of dike height



The structure height of a dike is composed of the following contributions; see also the Guidelines for Sea and Lake Dikes [TAW, 1999-2]:

- the reference level with a probability of being exceeded corresponding to the legal standard;
- the high water increase or lake level increase during the design period;
- the expected local ground subsidence during the design period;
- the bonus due to squalls, gusts, seiches and other local wind conditions;
- the expected decrease in crest height due to settling of the dike body and the undersoil during the design period;
- the wave run-up height and the wave overtopping height.

Contributions (a) to (d) cannot be influenced, whereas contribution (e) can be influenced. Contribution (f) also depends on the outer slope, which can consist of various materials, such as an asphalt layer, a cement-concrete dike covering (stone setting) or grass on a clay layer. A combination of these types is also possible. Slopes are not always straight, and the upper and lower slope may have different slopes if a berm has been applied. The design of a covering layer is not dealt with in this report. However, the aspects related to berms, slopes and roughness elements are dealt with when they have an influence on wave run-up and wave overtopping.

In this report the notation for symbols according to the Guidelines [TAW, 1989] is used as much as possible. The international symbol list is used only for wave height and wave period: the significant wave height is H_{m0} , the average wave period is T_m , and the spectral period is $T_{m-1,0}$. Furthermore, in the Guidelines the combined influence factor γ_B is used for the influence of a berm and/or angled wave attack. In this report, the two influences are distinguished by using γ_b for the influence of a berm and γ_β for the influence of the angle of wave attack.

The relative run-up is given by $z_{2\%}/H_{m0}$. The wave height H_{m0} is valid at the toe of the structure, as with the period $T_{m-1,0}$. In Chapter 5 of the above Guidelines it is described how the wave conditions, including H_{m0} , can be determined. For safety assessment, the conditions are

2. Wave run-up

given in the Hydraulic Boundary Conditions [RWS, 2001], and they may need to be converted for the parameters used here.

The relative run-up is usually given as a function of the surf similarity parameter, or breaker parameter, defined as:

$$\xi_0 = \frac{\tan \alpha}{\sqrt{s_0}} \quad (1)$$

where:

ξ_0	=	breaker parameter	(-)
α	=	angle of slope	(°)
s_0	=	wave steepness = $2 \cdot \pi \cdot H_{m0} / (g \cdot T_{m-1,0}^2)$	(-)
H_{m0}	=	wave height = $4 \cdot \sqrt{m_0}$	(m)
$T_{m-1,0}$	=	spectral wave period = m_{-1} / m_0	(s)
m_0	=	zero moment of spectrum	(m ²)
m_{-1}	=	first negative moment of spectrum	(m ² s)
g	=	acceleration of gravity	(m/s ²)

Various wave periods can be defined for a wave spectrum, in addition to the spectral period $T_{m-1,0}$, the peak period T_p (the period that gives the peak of the spectrum), the average period T_m (calculated from the spectrum or from the time signal) and the significant period $T_{1/3}$ (the average of the highest 1/3 part of the wave periods). The relationship T_p / T_m usually lies between 1.1 and 1.25, and T_p and $T_{1/3}$ are almost identical. In the Guidelines [TAW, 1989] the relationship $T_m = T_{1/3} / 1.15$ is used.

As described in section 1.3, the spectral period $T_{m-1,0}$ is a new parameter in the area of wave conditions. For any conversion of a known peak period for a single-peaked spectrum in not-too-shallow water (no 'flattened' spectrum) to the spectral period, the following factor can be used:

$$T_p = 1.1 \cdot T_{m-1,0} \quad (2)$$

For $\xi_0 < 2$ to 2.5 the waves break on the slope and this is usually the case with slopes flatter than 1:3. For larger values of ξ_0 the waves do not break on the slope. In that case the slopes are often steeper than 1:3 and/or the waves are characterised by a small wave steepness (e.g., swell). For heavy and very heavy breaking waves on a shallow foreshore large values of ξ_0 are also found. This is because the wave height is greatly reduced, whereas the wave period is not; this leads in some cases to a very small wave steepness.

2.2 General formula for wave run-up

The general formula that can be applied for wave run-up on dikes is given by:

$$z_{2\%} / H_{m0} = 1.75 \cdot \gamma_b \cdot \gamma_t \cdot \gamma_\beta \cdot \xi_0 \quad (3a)$$

with a maximum for larger ξ_0 of:

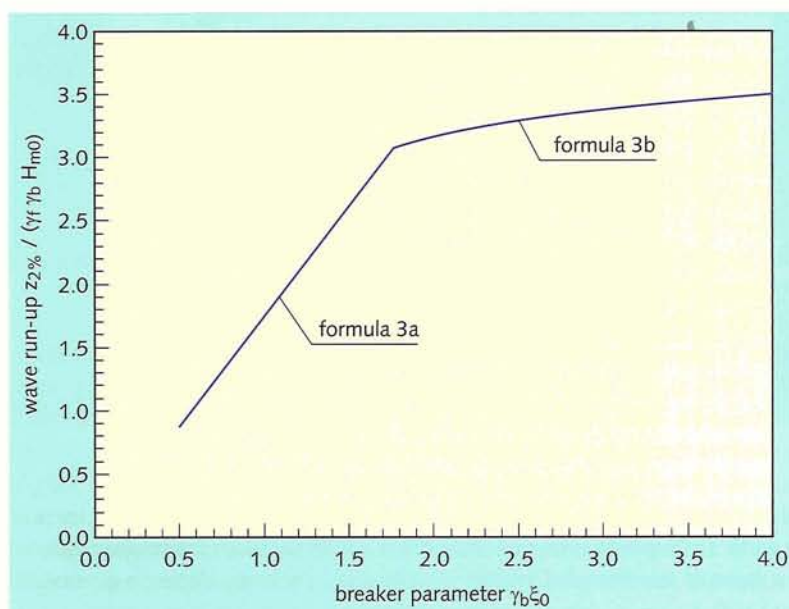
$$z_{2\%} / H_{m0} = \gamma_t \cdot \gamma_\beta \cdot (4.3 - 1.6 / \sqrt{\xi_0}) \quad (3b)$$

where:

$z_{2\%}$	=	2% wave run-up level above still water line	(m)
H_{m0}	=	significant wave height at toe of dike	(m)
ξ_0	=	breaker parameter (formula 1)	(-)
γ_b	=	influence factor for a berm	(-)
γ_f	=	influence factor for roughness elements on slope	(-)
γ_β	=	influence factor for angled wave attack	(-)

The formula is valid in the area $0.5 < \gamma_b \xi_0 < 8$ à 10 . The relative wave run-up $z_{2\%}/H_{m0}$ depends on the breaker parameter ξ_0 and three influence factors: for a berm (applied to the breaker parameter), roughness elements on the slope, and angled wave attack. Calculation of the influence factors is described later in this report.

Figure 3:
wave run-up as function of
breaker parameter



Formula 3 is shown in figure 3 in which the relative run-up $z_{2\%}/(\gamma_f \gamma_b H_{m0})$ is plotted against the breaker parameter $\gamma_b \xi_0$. Up to $\gamma_b \xi_0 \approx 1.8$ the relative run-up increases linearly with increasing $\gamma_b \xi_0$; for larger values, the increase slows towards an even less steep line. The theoretical maximum in formula 3b (for very large values of $\gamma_b \xi_0$, well outside the application area) is $4.3 \gamma_f \gamma_b$.

Large values of $\gamma_b \xi_0$ are found for relatively steep slopes and/or low wave steepness due to for example breaking on a shallow or very shallow foreshore. For very steep slopes and relatively deep water, formula 3b gives a rather conservative value and, in specific cases, further study is recommended. The theoretical limit value for a completely vertical structure is ($\xi_0 = \infty$) is: $z_{2\%}/H_{m0} = 1.4$, but this is well outside the application area examined.

In the Guidelines [TAW, 1989] a wave run-up formula for gently sloping (flatter than 1:2.5), smooth and straight slopes was given. After conversion, this becomes:

$$z_{2\%}/H_{m0} = 1.77 \cdot \xi_0 \quad (4)$$

This formula is almost identical to the linear formula 3a, except concerning the influence factors, and shows no levelling off for larger values of the breaker parameter, i.e., the wave run-

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up formula from the Guidelines is almost completely accepted and improved on specific points. For a design or assessment rule, it is advised not to follow the average trend. In many Dutch and international standards, a safety margin of one standard deviation is used, and this value is also supported by Vrouwenvelder [TNO, 1992]. This safety factor is also used in formulae 3.

For probabilistic calculations wave run-up can be calculated by:

$$z_{2\%}/H_{m0} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_0 \quad (5a)$$

with a maximum for larger ξ_0 of:

$$z_{2\%}/H_{m0} = \gamma_f \cdot \gamma_\beta \cdot (4.0 - 1.5 / \sqrt{\xi_0}) \quad (5b)$$

Although above formulae do not predict a perfect value of expectation, in the sense of statistics and based on measured points, the formulae are treated further in this report as the "average wave run-up".

The distribution around formula 5 can be described by a variation coefficient (standard deviation divided by the mean) in relation to this average line and is $V = \sigma / \mu = 0.07$.

Figures 4 - 6 show available measured points related to wave run-up. Each figure shows a specific part of the application area.

The measured points in figures 4 and 5 are limited to small-scale tests done by Van der Meer and De Waal [WL, 1993-2], on which the current report is based, on available large-scale measurements, that can be looked on as reliable, and finally on recent measurements with shallow and very shallow foreshores from Van Gent [WL, 1999-2].

Figure 4 is limited to smooth straight slopes under completely perpendicular wave attack and in relatively deep water (where waves do not often break). In these cases the breaker parameter is limited to a value of less than 4. Only for steeper and very steep slopes, e.g., steeper

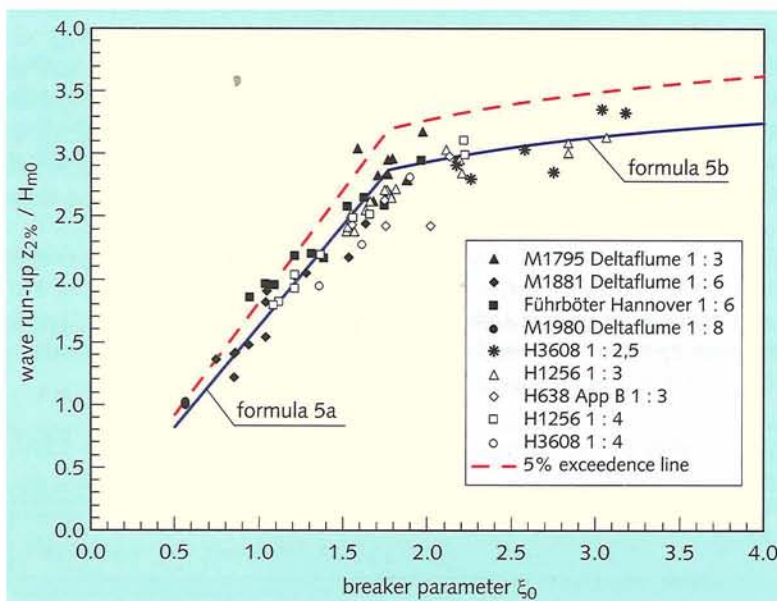


Figure 4:
wave run-up for smooth
straight slope in relatively
deep water with measured
points

than 1:2.5, greater values are found for the breaker parameter.

In figure 5 the data from figure 4 are shown again, together with the data for shallow and very shallow foreshores, for single- and double-peaked spectra in deep water before the foreshore. For a very shallow foreshore the wave steepness due to decrease in the wave height is very small and the breaker parameter is very large, even for flat slopes with 1:4 slope. The breaker parameter in figure 5 is therefore also given to a range of $\xi_0 = 10$.

Figure 5:
wave run-up for straight
smooth slopes including
shallow and very shallow
foreshores and double-peaked
spectra

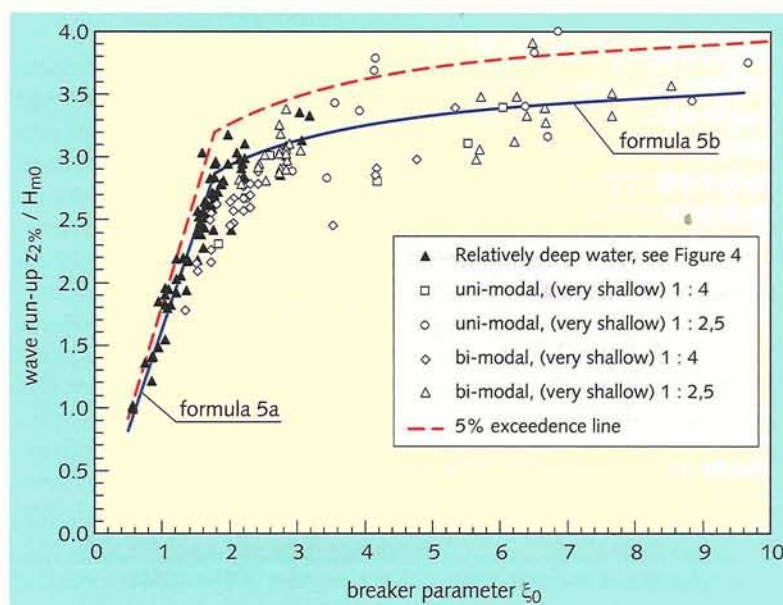
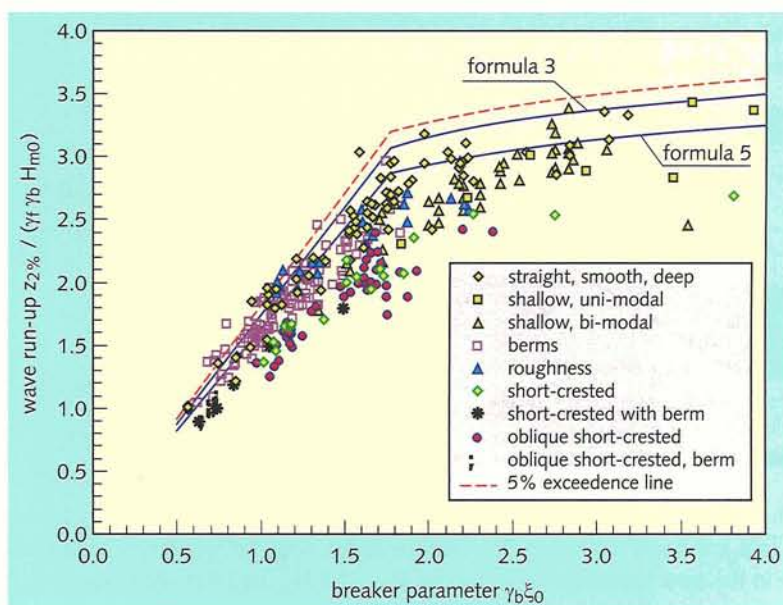


Figure 6 shows all available measured points including slopes with berms or roughness elements, and also including angled and short-crested wave attack. When all influences are brought together in a single figure, the scatter is greater than just for smooth straight slopes. This comes partly from the fact that when taking into account the influence factors some safety margin was included. The greater scatter is mainly in points that fall below the aver-

Figure 6:
wave run-up data including
possible influences



2. Wave run-up

age line. Above that, the scatter is almost identical to that shown earlier and $V = 0.07$ can be used. For this reason, in figures 4-6 only the upper 5% exceedance limit is shown, and not the lower one. Figure 6 shows both formula 3 and formula 5.

Formula 5 is not the formula that should be used for the wave run-up in deterministic design of dikes; then formula 3 should be used. Formula 5 can be used for probabilistic designs using the variation coefficient described above.

Each of the influence factors γ_b , γ_f and γ_β in formula 3 was established from experimental studies. A combination of influence factors is possible in the formula such that a very high total reduction (a low influence factor) is achieved. For example, a rubble mound slope with a maximally reducing berm under very oblique waves gives a total influence factor of about 0.24. This means that the wave run-up is one-quarter of that on a smooth slope without a berm with oblique wave attack. Because not all combinations of wave run-up reducing conditions have yet been studied, it is recommended that further research is needed if the influence factor becomes lower than 0.4:

$$\gamma_b \cdot \gamma_\beta \cdot \gamma_f \geq 0.4 \quad (6)$$

Finally, the simplest formula that has been used in the Netherlands for a long time, is:

$$z_{2\%} = 8 \cdot H_s \cdot \tan \alpha \quad (7)$$

This formula agrees with formula 3 for an average wave steepness of $s_0 = 0.048$, a value of 1.0 for all influence factors, and $\xi_0 < 1.8$.

2.3 Average slope

It often occurs that a dike slope does not consist of an entirely straight slope, but of sections with various slopes and often with one or more berms. Considering that the wave run-up formula requires a characteristic slope in the breaker parameter, a definition is required to combine the various slope sections. This definition for an average slope is given here, ignoring any berms. The influence of a berm is considered separately in section 2.6.

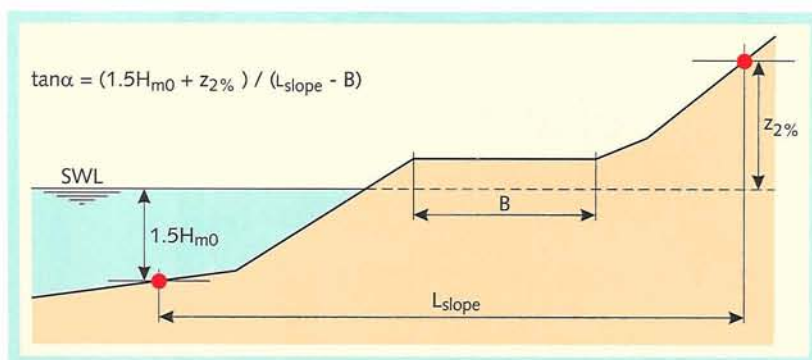


Figure 7:
determination of the characteristic slope for a cross-section consisting of various slope sections, excluding any berm influence

Figure 7 shows the definition diagram for this representative slope $\tan \alpha$ that is only based on slope sections and any berm is ignored. The representative slope for wave run-up $\tan \alpha$ is the average slope in the zone between the still water line - $1.5H_{m0}$ and still water line + $z_{2\%}$. Any berm present is not included for calculation of the average.

Since $z_{2\%}$ is unknown, it has to be determined by using an iterative method. The first estimate of $z_{2\%}$ is set at $1.5H_{m0}$. The average slope is then calculated between the points $1.5H_{m0}$ under and above the still water line, ignoring the influence of a berm. This is adequate for a manual calculation. It may occur that there is a large kink in the upper slope around $z_{2\%}$ (so-called "concave" and "convex" slopes). For these, the iterative method must be used for calculating the correct run-up value. This is therefore the recommended method using a computer. If $1.5H_{m0}$ or $z_{2\%}$ come above the crest level, then the crest height must be taken as the characteristic point.

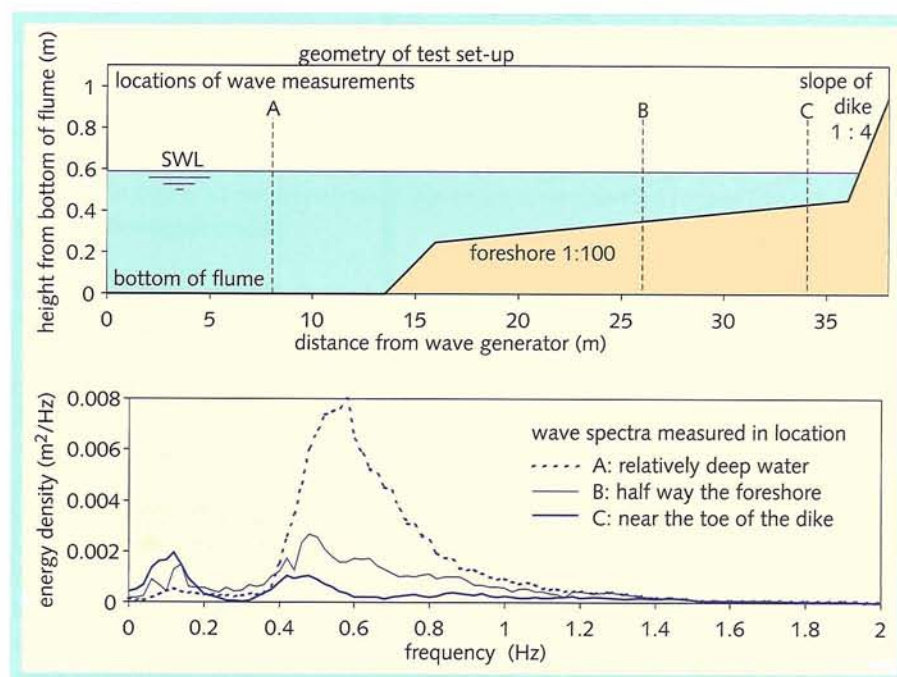
2.4 Influence of shallow foreshore

When waves reach a shallow foreshore they may break due to the limited depth. In principle this is favourable, because the wave height at the toe of the structure will therefore be lower, and this will apply also to wave run-up and wave overtopping.

In addition, the wave height distribution will also change. For relatively deep water at the toe of a dike ($h_m/H_{m0} > 3$ to 4) the probability of the wave heights follows a so-called Rayleigh distribution, for which h_m is the depth of water at the toe of the structure. For a shallow foreshore ($h_m/H_{m0} < 3$ to 4) the waves will break on the foreshore and the distribution will deviate from that in deep water, with especially the higher waves breaking, as shown diagrammatically in figure 8. For a Rayleigh distribution, the relationship $H_{2\%}/H_{m0} = 1.40$ holds, where $H_{2\%}$ is the wave height exceeded by 2% of the waves. For waves breaking on a foreshore this relationship is smaller and varies roughly between 1.1 and 1.4. For an extra influence factor for wave run-up in shallow water on a foreshore (in addition to the reduction of the wave height itself) it is advised to look for a relationship of $H_{2\%}/H_{m0}$.

Reality is in fact more complicated. The wave height H_{m0} is almost identical in deep water to $H_{1/3}$ (the average of the highest 1/3 of the waves). In shallow water, these wave heights can be very different.

Figure 8:
effect of shallow foreshore
on wave spectra



In the event of very heavy wave breaking a very shallow foreshore is considered. This is an application area in which a recent study was performed [WL, 1999-2], but not all the results of this study have yet been crystallised out. It is clear that for heavy breaking waves they show almost no signs of a spectrum with a well-defined peak period (the spectrum has been 'flattened') and that the spectral period $T_{m-1,0}$ is the obvious parameter.

Another aspect that plays a role for a very shallow foreshore is that very long waves (surf-beat) can occur due to the breaking. It is possible that this long wave energy is the cause of the relatively high run-up values for large values of the breaker parameter (mainly on the right side of figure 5). No study has yet been completed in this area. In figures 4 - 6 and formulae 3 and 5 account was taken of recent results from very shallow foreshores and the formula is therefore also applicable in this area.

2.5 Influence of the angle of incidence of wave attack

The angle of incidence of wave attack β is defined as the angle between the direction of propagation of the waves and the perpendicular to the long axis of the dike, see figure 9. Perpendicular wave attack is thus shown by $\beta = 0^\circ$. The angle of wave attack is the angle after any change of direction of the waves on the foreshore due to refraction.

The influence factor for the angle of wave attack is given by γ_β . Until recently little research had been done on oblique wave attack and the research that had been carried out related to long-crested waves, which have no directional distribution. The wave crests thus lie equally apart from each other. In model studies with long-crested waves, the wave crest is as long as the wave machine and the wave crests are equally spaced apart. In nature, waves are short-crested, which means that the wave crests have a certain length and the waves have a certain main direction. The individual waves have a direction around this main direction.

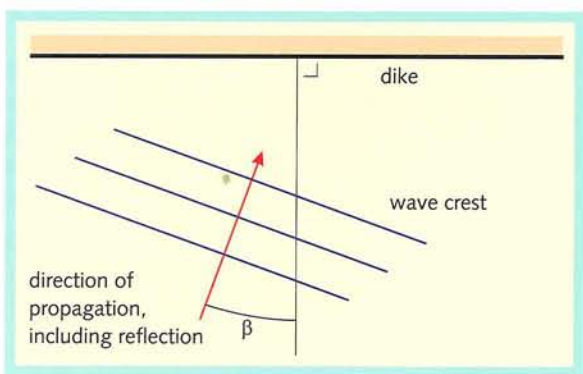


Figure 9:
definition of angle of
wave attack

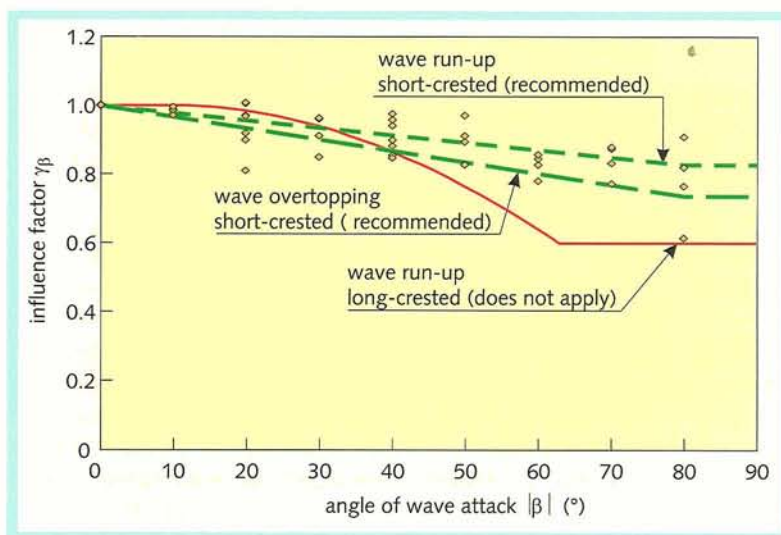
The amount of variation around this main direction (directional distribution) can be described by a certain scatter. Only long swell waves, such as from the ocean, have such long crests that one can speak of long-crested waves. A wave-field in a strong wind is short-crested.

In the report of Van der Meer and De Waal [WL, 1990] a study is described into wave run-up and wave overtopping in which the influence of angled attack and directional scatter was examined. Figure 10 shows a summary of the study results as discussed in Van der Meer and De Waal [WL, 1993-2]. The influence factor γ_β is plotted against the angle of wave attack, $|\beta|$.

Long-crested waves cause between $0^\circ < |\beta| < 30^\circ$ almost the same wave run-up as perpendicular wave attack. After that, the influence factor falls quite quickly to about 0.6 at 60° . For short-crested waves, the angle of wave attack has clearly less influence, mainly because within the concentrated wave-field the individual waves deviate from the main direction β . For both wave run-up and wave overtopping (see Chapter 3), the influence factor for short-crested waves decreases to a certain value at about 80° to 90° . This value is $\gamma_\beta = 0.8$ for 2% run-up and 0.7 for wave overtopping. For very oblique waves the influence factor is therefore a minimum of 0.7 to 0.8 and not 0.6 as found for long-crested waves.

Considering that a wave-field under storm conditions can be regarded as short-crested, it is recommended to use the lines in figure 10 for short-crested waves.

Figure 10:
influence factor γ_β for angle
of wave attack with measured
points for run-up for short-
crested waves



For 2% wave run-up and for wave overtopping different influence factors apply during angled wave attack, because the incoming wave energy per linear metre of the structure for angled wave attack is less than for perpendicular attack. Wave overtopping is defined as a discharge per linear metre of the structure whereas run-up does not depend on the length of the structure.

The lines in figure 10 for short-crested waves are recommended for use and can be described by the following formulae:

For 2% wave run-up with short-crested waves:

$$\begin{aligned}\gamma_\beta &= 1 - 0.0022 |\beta| & (0^\circ \leq |\beta| \leq 80^\circ) \\ \gamma_\beta &= 1 - 0.0022 \cdot 80 & (|\beta| > 80^\circ)\end{aligned}\quad (8)$$

For wave overtopping with short-crested waves:

$$\begin{aligned}\gamma_\beta &= 1 - 0.0033 |\beta| & (0^\circ \leq |\beta| \leq 80^\circ) \\ \gamma_\beta &= 1 - 0.0033 \cdot 80 & (|\beta| > 80^\circ)\end{aligned}\quad (9)$$

In practice waves that are at an angle of more than 80° to the perpendicular can occur, or the wave direction can even come from the land. This must finally reduce wave run-up and wave overtopping to zero. It has been decided to adjust the wave height and period and not

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the influence factor. For $80^\circ < |\beta| \leq 110^\circ$ the wave height H_{m0} and the wave period $T_{m-1,0}$ are adjusted as follows:

- H_{m0} is multiplied by $\frac{110-|\beta|}{30}$
- $T_{m-1,0}$ is multiplied by $\sqrt{\frac{110-|\beta|}{30}}$

For $110^\circ < |\beta| \leq 180^\circ$ then $H_{m0} = 0$, which results in wave run-up $z_{2\%} = 0$ and wave overtopping $q = 0$.

2.6 Influence of berms

Figure 11 shows diagrammatically an example of a dike with a berm. The middle of the berm lies at a depth d_h below the still water line. The slope of the berm in the Netherlands is often 1:15. The width of the berm is given by B , which is the horizontal distance between the front and rear of the berm; a definition of a berm is given in section 1.2.

The slope of a berm must lie between horizontal and 1:15 and the width of a berm should not exceed one-quarter of the wavelength. If the berm does not conform to these conditions then wave-run-up and wave overtopping must be determined by interpolation between the steepest berm (1:15) and a gentle slope (1:8) in the one case, or by interpolation between the longest possible berm ($0,25L_0$) and a foreshore in the other case. For calculations of wave run-up and wave overtopping, an angled berm is first drawn to a horizontal berm, as shown in figure 11. Then the lower and upper slopes are drawn. The berm width B to be taken into account is therefore shorter, whereas the berm depth, d_h , remains the same in relation to the still water line.

The influence factor γ_b that can be taken into account for a berm consists of two factors: one for the influence of the width of the berm, r_B , and one for the position of the middle of the berm in relation to the still water line, r_{dh} .

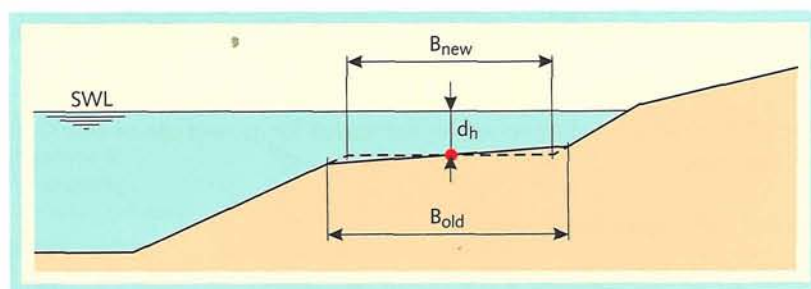


Figure 11:
diagram of width and depth
of berm

The following applies:

$$\gamma_b = 1 - r_B \cdot (1 - r_{dh}) \text{ waarbij } 0.6 \leq \gamma_b \leq 1.0 \quad (10)$$

If the berm lies on the still water line then $r_{dh} = 0$ and r_B ensures that r_B is less than 1 (the influence of the berm width). If the berm does not lie on the still water line, r_B is multiplied by a number less than 1 and the influence factor γ_b is again larger than in the case that the berm lies on the still water line.

Influence of berm width r_B

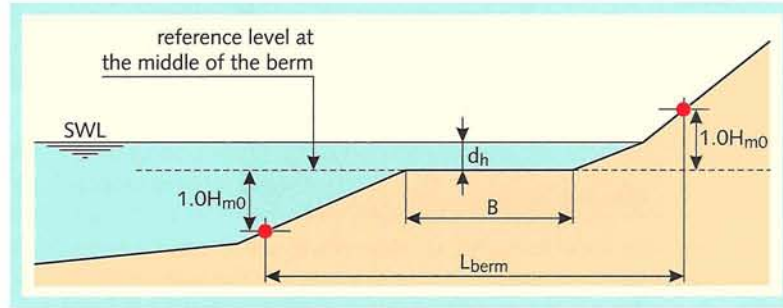
The influence of berm width can be found by examining the change in the slope, see figure 12:

$$r_B = 1 - \frac{2 \cdot H_{m0} / L_{berm}}{2 \cdot H_{m0} / (L_{berm} - B)} = \frac{B}{L_{berm}} \quad (11)$$

Influence of berm depth r_{dh}

The position of the berm in relation to the still water line has of course an influence on wave run-up. The berm is most effective when close to the still water line. The influence of the berm disappears when the berm lies higher than the run-up on the lower slope; the run-up does not then reach the berm and we can actually talk of run-up on a slope without a berm. It is also suggested that the influence of the berm disappears when it lies more than $2 \cdot H_{m0}$ under the still water line.

Figure 12:
determination of changes in
slope for berm



The influence of the berm position must be described over the space between $2 \cdot H_{m0}$ under the still water line up to $z_{2\%}$ on the lower slope. This influence is shown in figure 13, using a calculation example of a 1:3 slope. The berm position d_h/H_{m0} is plotted on the horizontal axis against the total influence factor for a berm, γ_b , see formula 10.

The influence of the berm position can be determined using a cosine function, in which the cosine is given in radians by:

$$r_{dh} = 0.5 - 0.5 \cdot \cos \left(\pi \frac{d_h}{x} \right) \quad (12)$$

where:

$$\begin{aligned} x &= z_{2\%} \text{ if } z_{2\%} > -d_h > 0 && \text{(berm above still water line)} \\ x &= 2 \cdot H_{m0} \text{ if } 2 \cdot H_{m0} > d_h \geq 0 && \text{(berm below still water line)} \\ r_{dh} &= 1 \text{ if } -d_h \geq z_{2\%} \text{ or } d_h \geq 2 \cdot H_{m0} && \text{(outside influence area)} \end{aligned}$$

The influence of a berm can be written in full from formulae 10 to 12 as:

$$\gamma_b = 1 - \frac{B}{L_{berm}} \left(0.5 + 0.5 \cdot \cos \left(\pi \frac{d_h}{x} \right) \right) \text{ with } 0.6 \leq \gamma_b \leq 1.0 \quad (13)$$

This means that the influence of a berm is at a maximum for $d_h = 0$, and then $\gamma_b = 1 - B/L_{berm}$ (see also figure 13). This is actually valid for identical upper and lower slopes. If the upper and lower slopes have different slopes then the berm position with the maximum influence can deviate somewhat from the still water line.

In figure 13 lines are shown for various berm widths, B/H_{m0} . For a given wave period, the

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berm width B/L_0 can also be used. Overall, this means that $B/H_{m0} = 10$ has the same size as $B/L_0 = 0.25$, which is the greatest width that exists for the present definition of a berm. The greater the width, the greater the influence of the berm. The maximum influence is actually always limited to $\gamma_b = 0.6$.

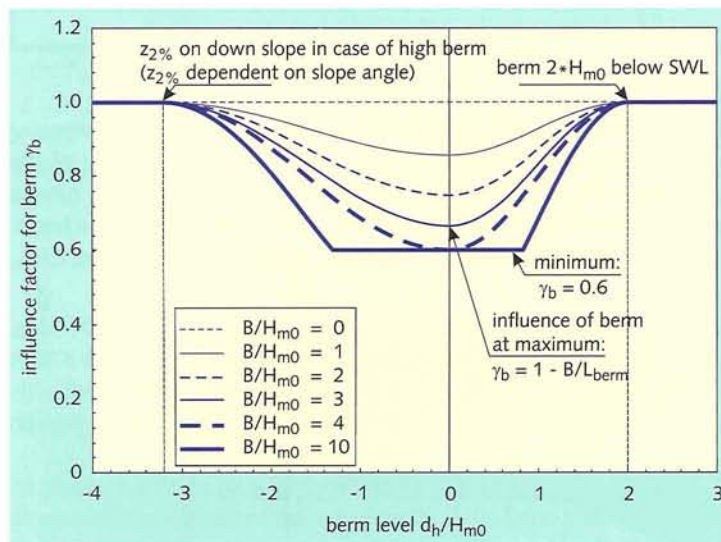


Figure 13:
influence factor for influence
of berm

The berm is most effective if it lies on the still water line ($r_{dh} = 0$) and the berm width is optimal when the influence factor reaches 0.6. In principle these formulae can be used to determine the berm width for every geometry of a dike (with a berm). For a berm on the still water line, the optimal berm width is (see also formula 13):

$$B = 0.4 \cdot L_{berm} \quad (14)$$

For the calculation of wave run-up, in the case of a relatively high-lying berm, a check must be made as to whether the calculated wave run-up level does actually reach the front of the berm. This check must take place when taking into account any influence from roughness elements, angled incoming waves and the lower lying berms already taken into account.

Finally, it is possible that there is more than one berm present in one dike profile. The influence factors must then be combined from low to high, to be determined with a minimum of 0.6, unless the collective berm width is greater or much greater than $0.25 \cdot L_0$.

2.7 Influence of roughness elements

The influence of roughness elements on wave run-up is given by the influence factor γ_r . In Appendix 11 of the Guidelines [TAW, 1989], a table is shown of influence factors for various sorts of slope protection. The origin of most of the data from this table can be found in the Russian study with regular waves, from the 1950s. This table was developed in the report by TAW [TAW, 1972] and has found its way into various international manuals.

New and often large-scale studies with irregular waves have led to a new table for influence factors for slopes with some or no roughness elements. These reference types (established on the basis of research) together with the accompanying influence factors for roughness elements are:

Reference type	γ_f
Concrete	1.0
Asphalt	1.0
Closed concrete block	1.0
Grass	1.0
Vilvoorden stone	0.85
Basalt	0.90
Haringman	0.90
Fixtone - open stone asphalt	0.90
Armorflex	0.90
Small blocks over 1/25 of surface	0.85
Small blocks over 1/9 of surface	0.80
1/4 of block revetment 10 cm higher	0.90
Ribs (optimum dimensions)	0.75
Armour rock - two layers thick	0.55
Armour rock - single layer	0.70

Artificial roughness elements on the slope (blocks and ribs) and the roughness of armour rock are discussed in more detail later.

Furthermore, based on the above table, an influence factor has been estimated for almost all types of slope that occur in the Netherlands [DWW, 2002]. The end result is a table showing a complete overall view of influence factors and this table is included in this report as Appendix 1.

The values given for the influence factor γ_f apply in principle for wave heights greater than about 0.75 m. It is possible that a relatively larger hydraulic roughness exists for smaller wave heights, which would lead to a lower influence factor. When this influence is known to occur, such as for example with grass (see "Technical report on erosion resistance of grassland dike cover" [TAW, 1998-2]), then these lower values are used.

The influence factors apply for $\gamma_b \xi_0 < 1.8$. From $\gamma_b \xi_0 = 1.8$ the influence factor increases linearly up to 1 for $\gamma_b \xi_0 = 10$, and it remains 1 for any greater values. The influence factors apply when at least the part between $0.25 \cdot z_{2\%,smooth}$ under and $0.5 \cdot z_{2\%,smooth}$ above the still water line is covered with roughness elements. For smaller areas with roughness elements and composite slopes, the procedure is described later in this report. This means that $z_{2\%,smooth}$ is the wave run-up for a smooth slope.

Roughness elements on slope

A reasonable amount of research has been performed on slopes on which roughness elements, such as blocks and ribs, have been placed. The width of a block or rib is given by f_b , the height by f_h and the distance between ribs by f_L . The position of the blocks is determined by the part of the total slope surface that will be covered by the blocks. The optimal distance between ribs is $f_L/f_b = 7$. For application of the influence factors below, f_h/f_b must be between 5 and 8. When the total surface is covered by blocks or ribs and when the height is at least $f_h/H_{m0} = 0.15$, then the following minimum influence factors are found:

Block, 1/25 of total surface covered	$\gamma_{f,min} = 0.85$
Block, 1/9 of total surface covered	$\gamma_{f,min} = 0.80$
Ribs, $f_L/f_b = 7$ apart (optimal)	$\gamma_{f,min} = 0.75$

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A greater block or rib height than $f_h/H_{m0} = 0.15$ has no further reducing effect. If the height is less, then one can interpolate linearly towards $\gamma_f = 1$ for $f_h/H_{m0} = 0$:

$$\gamma_f = 1 - (1 - \gamma_{f,min}) * (f_h/H_{m0})/0.15 \text{ for } f_h/H_{m0} < 0.15 \quad (15)$$

As for the influence factors in Appendix 1, the factors in formula 15 apply for $\gamma_b \xi_0 < 1.8$ and increase linearly up to 1 for $\gamma_b \xi_0 = 10$.

Armour rock slopes

A large number of tests have been performed for slopes armoured with a two diameters thick layer of rock on an impermeable under-soil or core. These are shown in figure 14, together with the average trend for smooth straight slopes, formula 5. The wave run-up reducing effects of armour rock is bigger than that on smooth slope, especially when the value of the breaker parameter is low. For large values of the breaker parameter (larger than about 10), run-up is similar for the armour rock and smooth slopes.

If a value of $\gamma_f = 0.55$ is used for an armour rock slope for $\xi_0 < 1.8$ and thereafter a linear increase between $1.8 < \xi_0 < 10$ to $\gamma_f = 1.0$, then the dashed line in figure 14 is used. This linear increase between $1.8 < \xi_0 < 10$ is also used for other influence factors for roughness elements.

Roughness elements do not always occur across the entire slope, but only over a part. The influence factor applies for that part of the slope, but it is not the one that may be applied for determination of wave run-up and wave overtopping. The procedure for combining various roughness elements is described in section 2.9.

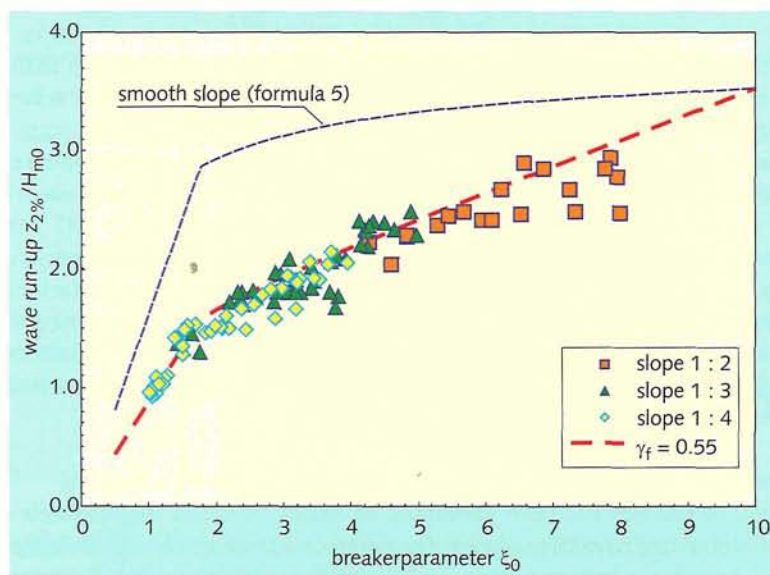


Figure 14:
wave run-up on armour rock
slope (with impermeable
under-layer)

2.8 Influence of vertical or very steep wall on slope

In some cases a vertical or very steep wall is placed on the top of a slope to reduce wave overtopping. A relatively small wall and not large vertical structures such as caissons and high walls on quays, is considered. The wall must form an essential part of the slope, and sometimes including a berm.

The influence factors for a vertical or steep wall apply for the following studied application area:

- The average slope of $1.5.H_{m0}$ below the still water line to the foot of the wall (excluding a berm) must lie between 1:2.5 to 1:3.5.
- The width of all berms together must be no more than $3.H_{m0}$.
- The foot of the wall must lie between about $1.2.H_{m0}$ under and above the still water line;
- The minimum height of the wall (with a high foot) is about $0.5.H_{m0}$. The maximum height (for a low foot) is about $3.H_{m0}$.

Other vertical walls can be calculated using the report "Wave run-up and forces on vertical water defences" [WL, 1998].

It is possible that work will be performed to prepare guidelines for wave run-up and wave overtopping for vertical structures, in the future. Until then the influence factors below can be used within the application area described. Wave run-up formulae are given for a completely vertical wall in the new Guideline for hydraulic structures [TAW, 2002]. Formula 23 from Chapter 3 gives the same formula as in the Guideline with a factor of 3.0 instead of 2.3 and a factor of 0.13 instead of 0.2.

The wave run-up formulae apply for a slope of slope 1:1 or flatter. A steep wall is thus defined as a wall steeper than 1:1. In this sort of case, wave run-up is less important - especially for a vertical wall - than wave overtopping. The influence factor γ_v that must be applied for wave overtopping is therefore described here.

For wave overtopping (see Chapter 3) a breaker parameter ξ_0 is required, as for wave run-up. A vertical wall soon leads to a large value for the breaker parameter when determining an average slope as described in section 2.3, figure 7. This means that the waves will not break. The wall will be on top of the slope, possibly even above the still water line, and the waves will break on the slope before the wall. In order to maintain a relationship between the breaker parameter and the type of breaking on the slope, the steep or vertical wall must be drawn as a slope 1:1 when determining the average slope. This slope starts at the foot of the vertical wall. The average slope and the influence of any berm must be determined with a 1:1 slope instead of the actual steep slope or vertical wall, according to the procedure given in section 2.3.

Furthermore, the wave overtopping for a vertical wall on a slope is smaller than for a 1:1 slope on top of a dike profile. The influence factor for a vertical wall on a slope is $\gamma_v = 0.65$. For a 1:1 slope, this influence factor is $\gamma_v = 1$. Interpolation must be performed for a wall that is steeper than 1:1 but not vertical:

$$\gamma_v = 1.35 - 0.0078 \cdot \alpha_{wall} \quad (16)$$

where α_{wall} is the angle of the steep slope in degrees (between 45° for a 1:1 slope and 90° for a vertical wall).

2.9 Interpolations between slopes, berms, foreshores and various roughness elements

Definitions are given in section 1.2 for a slope, a berm and a foreshore and the wave run-up and wave overtopping formulae apply for these definitions. For a dike profile that does not completely conform to these definitions, run-up and wave overtopping can be established via inter-

2. Wave run-up

polation, as described in the procedures in this section. A slope can also consist of slope sections with various roughness elements and procedures are also given that cover these eventualities.

Slope between 1:8 and 1:15

A slope is defined to a slope of 1:8 and a berm must not be steeper than 1:15, whereby the actual slope of the berm no longer has any influence. Continuous slopes between 1:8 and 1:15, thus without more steep slope sections, can be initially treated as normal slopes, but the reliability of the results will be less than for steeper slopes.

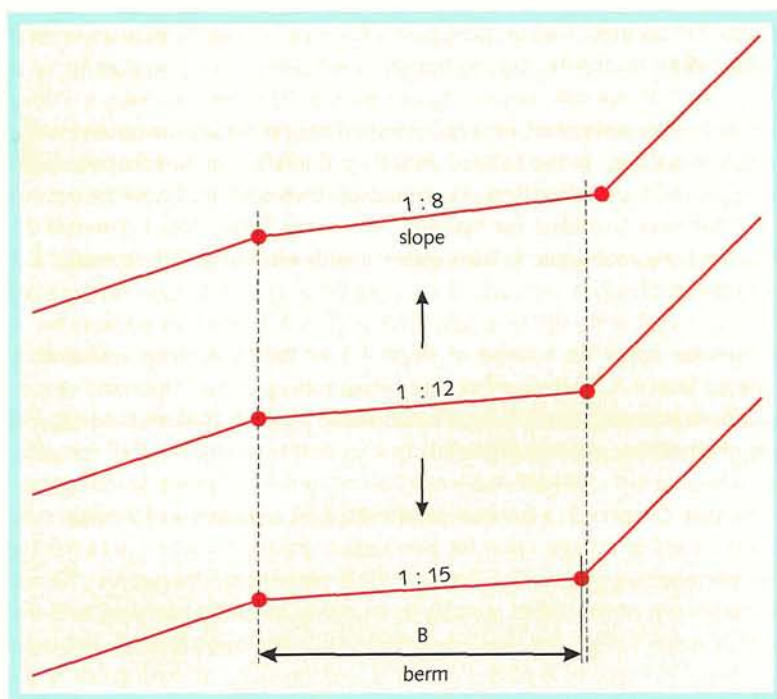


Figure 15:
determination of slope and
berm for slope section with
slope between 1:8 and 1:15

Slopes or parts of slopes with a slope between 1:8 and 1:15 and with a horizontal length limited to a maximum of $0.25.L_0$ lie almost exactly between the definitions of a slope and a berm. Determination of the run-up then proceeds as follows (see also figure 15):

- Draw the profile from the front of the gently sloping slope section (the seaward side) with a slope section of 1:8 until it intersects with the original upper slope.
- Determine run-up/wave overtopping for the 1:8 slope section as if it was a slope with only slope sections.
- Draw the 1:15 berm from the front of the gently sloping slope section. Considering that the berm is flatter than the gently sloping slope, an intersect is always found with the connecting slope section above the gently sloping slope if this is drawn downwards. Then make the berm horizontal according to figure 11.
- Determine run-up/wave overtopping as if it was a slope with a berm. (Note: if there is a berm before or after on the slope, then the berms must be combined and checked if the total horizontal width is not greater than $0.25.L_0$. If this is the case, then the appropriate slope to be drawn should be calculated using the procedure for a berm wider than $0.25.L_0$.)
- Interpolate with the average slope ($\tan \alpha$) as parameter between the two values found above:

$$Z_{2\%} = Z_{2\% \text{ 1:8}} + (Z_{2\% \text{ berm}} - Z_{2\% \text{ 1:8}}) * (1/8 - \tan \alpha) / (1/8 - 1/15) \quad (17)$$

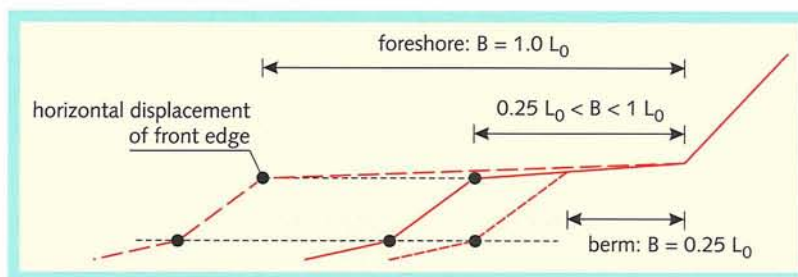
where $z_{2\%}$ is the value of the run-up. The procedure for wave overtopping is given in section 3.3.

Berm wider than $0.25 L_0$

If a berm is wider or much wider than one-quarter of the wave length, the influence factor will eventually be smaller than the minimum value of $\gamma_b = 0.6$ established in section 2.6. Therefore a requirement is also set for the maximum width of the berm. Waves on a foreshore must have sufficient length to adjust to the depth of the foreshore. Therefore a requirement for a foreshore is that it must have a minimum length of at least one wave length. A berm that is longer than $0.25 L_0$, but shorter than $1 L_0$ is thus exactly between the definitions of a berm and a foreshore. Wave run-up and wave overtopping can then be determined by interpolation.

Figure 16 shows a diagrammatic example of how the actual profile is converted into a berm with a width of one-quarter of a wave length and to a foreshore with the length of one full wave length.

Figure 16:
determination of foreshore
and berm for slope section
with length greater than
 $0.25 L_0$



The remaining procedure is then:

- Determine wave run-up for a berm of length $0.25 L_0$.
- Determine wave run-up for a foreshore drawn according to figure 16 to a length of $1 L_0$. This means that the significant wave height at the position of the new toe of the structure must be determined. In this case the new toe of the structure is the start of the slope above the foreshore. A simple determination of this wave height is: $H_{m0} \leq 0.5 \cdot \text{water depth}$. If the original H_{m0} is smaller than half the water depth, then H_{m0} remains unchanged. In the other case H_{m0} is the same as half the water depth. Using the calculated lower wave height, the wave run-up on the upper slope is calculated. The wave period and the angle of wave attack remain unchanged.
- Interpolate between the two calculated wave run-up heights using B/L_0 as parameter:

$$z_{2\%} = z_{2\%,\text{berm}} - (z_{2\%,\text{berm}} - z_{2\%,\text{foreshore}}) * (B/L_0 - 0.25)/0.75 \quad (18)$$

- If the 'too long' berm is positioned very high, for example above the still water line, then $z_{2\%,\text{foreshore}}$ is presumed to be the rear side of the berm (the start of the upper slope).

The above procedure only applies for wave run-up. A more complicated procedure needs to be followed for wave overtopping, as explained in section 3.3. It is possible that by drawing a foreshore, no wave overtopping will occur because the wave height is reduced so significantly, which means that interpolation would no longer be possible.

Slopes with composite roughness elements

Slopes with roughness elements over the entire surface will not always occur, but only over part of the slope. The influence factor applies for that section of the slope, but this is not the influence factor that may be applied in the wave run-up and wave overtopping formulae. Using automated calculations an influence factor for roughness elements can be given for

each section of the slope, but for calculation of run-up or wave overtopping a weighted influence factor must be determined. From a Polish study especially, performed on contract to the Directorate-General for Public Works & Water Management and as a follow-up to studies in the Netherlands, a procedure could be established for determining this weighted influence factor for roughness elements [WL, 1997-2].

It appears that roughness elements applied only under the still water line have no effect and in that case it is looked on as a smooth slope. If the same roughness elements exist above the still water line then the weighted average is determined over the slope that lies between $0.25 \cdot z_{2\%,smooth}$ under and $0.5 \cdot z_{2\%,smooth}$ above the still water line, in which $z_{2\%,smooth}$ is the wave run-up on a smooth slope, with consideration of any influence as a consequence of angled wave attack and berms. Roughness elements above $SWL + 0.5 \cdot z_{2\%,smooth}$ have little or no effect.

The above procedures can lead to a discontinuity in the case that the roughness elements lie from the still water line under water (thus, no influence). When the roughness elements are drawn just above the still water line, then the total roughness is taken into account (influence over $0.25 \cdot z_{2\%,smooth}$ over the part under water). Therefore the following extra condition is given for roughness elements above and below the still water line: the influence factor to be taken into account under the still water line may not be less than the influence factor above the still water line.

Weighting of the various influence factors occurs by including the lengths of the appropriate sections of the slope (between $SWL - 0.25 \cdot z_{2\%,smooth}$ and $SWL + 0.5 \cdot z_{2\%,smooth}$). If within the above established limits three slopes occur with lengths of L_1 , L_2 and L_3 and influence factors for the roughness elements of $\gamma_{i,1}$, $\gamma_{i,2}$ and $\gamma_{i,3}$, respectively, then the weighted average is:

$$\gamma_i = \frac{\gamma_{i,1} \cdot L_1 + \gamma_{i,2} \cdot L_2 + \gamma_{i,3} \cdot L_3}{L_1 + L_2 + L_3} \quad (19)$$

Because roughness elements are only effective in a limited area, the full influence can be achieved by only applying roughness elements in this area. The costs will therefore be less than when covering the entire slope with roughness elements.

Berms in a direct even line with various roughness elements

If two berms are lying in a straight line it is recommended that the berms are combined together into one long berm. If the roughness elements are different between the two berms, then a weighted influence factor for the roughness elements must be calculated:

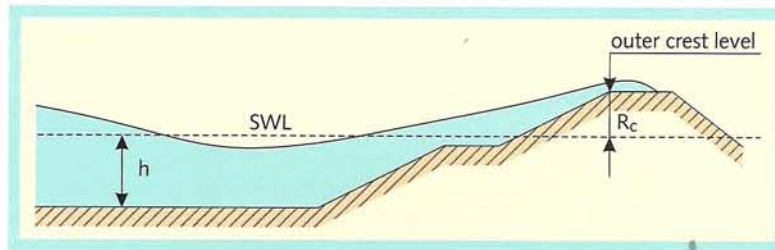
$$\gamma_{i,long\ berm} = \frac{\gamma_{i,berm1} \cdot L_{berm1} + \gamma_{i,berm2} \cdot L_{berm2}}{L_{berm1} + L_{berm2}} \quad (20)$$

3. Wave overtopping

3.1 Average wave overtopping discharge

For wave overtopping the crest height is lower than the wave run-up levels of the highest waves. The parameter that must then be used is the free crest height R_c , see figure 17.

Figure 17:
free crest height for wave
overtopping



This is the difference in height between the still water line and the crest height. The crest height itself can be given as the dike-table height h_d , determined in relation to for example Normal Amsterdam Water Level (NAP). The crest height is determined at the position of the outer crest line (and therefore not in the middle of the crest). The dike-table height decreased by the corrected water level (also in relation to NAP) then gives the free crest height R_c (see definitions in section 1.2).

Wave overtopping is usually given as an average discharge per linear metre of width, q , for example in m^3/s per m or in l/s per m. The Guidelines [TAW, 1989] show that for relatively heavy sea conditions with waves several metres high, the 2% run-up used provides a wave overtopping discharge in the order of $1 l/s$ per m. This is about $0.1 l/s$ per m for low waves, such as in the Dutch Large Rivers area. If we accept in the Large Rivers area $1 l/s$ per m, this gives a reduction for the required height (taking into consideration a minimum crest freeboard of 0.50 m). The Guidelines also say: "Whichever criterion is applicable also depends on the structure of the dike and any buildings. In certain cases, such as with a protected crest and inner slope, when water enters, sometimes $10 l/s$ per m can be used". The Guidelines suggest that the following average discharges are indicative for erosion of the inner slope:

- $0.1 l/s$ per m for sandy soil with a poor grass cover;
- $1.0 l/s$ per m for clayey soil with a reasonably good grass cover;
- $10 l/s$ per m for a clay covering and a grass cover according to the requirements for the outer slope or for a armoured inner slope.

Research is ongoing to substantiate better the relationship between wave overtopping and the capacity of the inner slope. A method is also given in the Guideline on Safety Assessment [TAW, 1999-1].

Wave overtopping can be described in two formulae linked to each other: one for breaking waves ($\gamma_b \xi_0 < \approx 2$), where wave overtopping increases for increasing breaker parameter ξ_0 , and one for the maximum that is achieved for non-breaking waves ($\gamma_b \xi_0 > \approx 2$).

Figure 18 shows an example of the result of these wave overtopping formulae. As for wave run-up, the breaker parameter ξ_0 is plotted on the horizontal axis. Instead of relative wave run-up, now a dimensionless wave overtopping discharge is plotted on the vertical logarithmic axis. Three lines are shown for three different relative crest heights R_c/H_{m0} (one, two and three times a wave height above the still water line). In the example in figure 18 a 1:3 smooth and straight slope is assumed, with perpendicular wave attack.

3. Wave overtopping

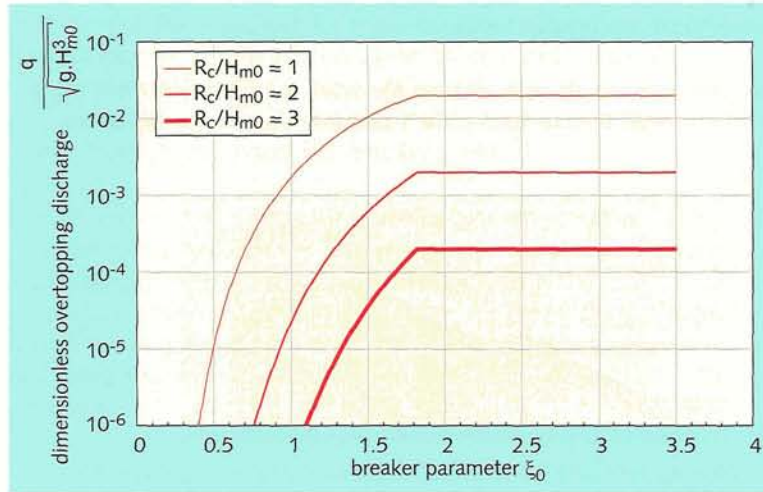


Figure 18:
wave overtopping as function
of breaker parameter (1:3
slope)

The wave overtopping formulae are exponential functions with the general form:

$$q = a \cdot \exp(b \cdot R_c) \quad (21)$$

The coefficients a and b are still functions of the wave height, slope angle, breaker parameter and the influence factors described in Chapter 2. The complete formulae are:

$$\frac{q}{\sqrt{g H_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} \gamma_b \cdot \xi_0 \cdot \exp \left(-4.3 \frac{R_c}{H_{m0}} \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \quad (22)$$

and a maximum of:
$$\frac{q}{\sqrt{g H_{m0}^3}} = 0.2 \cdot \exp \left(-2.3 \frac{R_c}{H_{m0}} \frac{1}{\gamma_f \cdot \gamma_\beta} \right) \quad (23)$$

where:

q	= average wave overtopping discharge	(m ³ /s per m)
g	= acceleration due to gravity	(m/s ²)
H_{m0}	= significant wave height at toe of dike	(m)
ξ_0	= breaker parameter = $\tan \alpha / \sqrt{s_0}$	(-)
s_0	= wave steepness = $2 \cdot \pi \cdot H_{m0} / (g \cdot T_{m-1,0}^2)$	(-)
$T_{m-1,0}$	= spectral wave period at toe of dike	(s)
$\tan \alpha$	= slope, see figure 7	(-)
R_c	= free crest height above still water line	(m)
γ	= influence factors for influence of berm, roughness elements, angle of wave attack, and vertical wall on slope, see Chapter 2	(-)

The dimensionless wave overtopping discharge $q / \sqrt{g H_{m0}^3}$ and the relative crest height R_c / H_{m0} are both related to the breaker parameter and/or the slope of the structure. In order to take into account the influence of different conditions, the dimensionless crest height is apparently increased by dividing by the influence factors γ_b , γ_f , γ_β , γ_v described in Chapter 2. With one exception, as described in this chapter, the formulae from Chapter 2 apply the influence factors.

Both design formulae 22 and 23 are shown diagrammatically in figures 19 and 20. The dimensionless wave overtopping discharge on the vertical axis in figure 19 is given by:

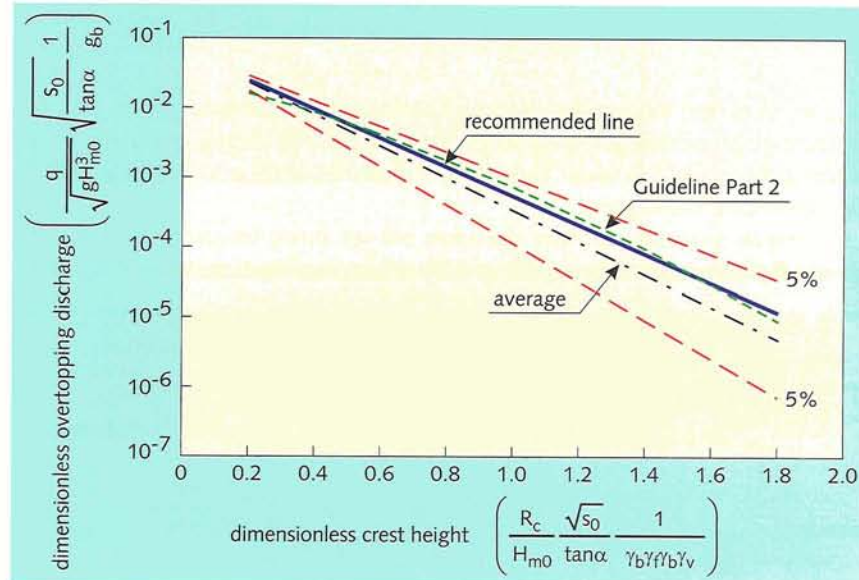
$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} \cdot \frac{\sqrt{\tan \alpha}}{\gamma_b \cdot \xi_0}$$

and the dimensionless crest height by:

$$\frac{R_c}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_t \cdot \gamma_\beta \cdot \gamma_v}$$

In both figures the recommended lines are shown together with a mean with 5% lower and upper exceedance limits, based on measurements (see later). The formula from the Guidelines [TAW, 1989] is also shown, which agrees almost exactly with the new recommended line.

Figure 19:
wave overtopping with
breaking waves



Wave overtopping for non-breaking waves is no longer dependent on the breaker parameter. The formula for breaking waves (formula 22) is valid up to the maximum, which is in the region of $\gamma_b \xi_0 = 2$. A check must still be made as to whether formula 22 exceeds the maximum of formula 23.

Generally it can be concluded that for wave run-up and wave overtopping on smooth straight slopes the differences with the Guidelines are very small. The new formulae take into account the fact that a maximum is reached for non-breaking waves. Improvement is mainly in the description of the reliability of the formulae (see later) and the better description of the influence of berms, roughness elements, angle of wave attack and vertical walls on a slope.

3. Wave overtopping

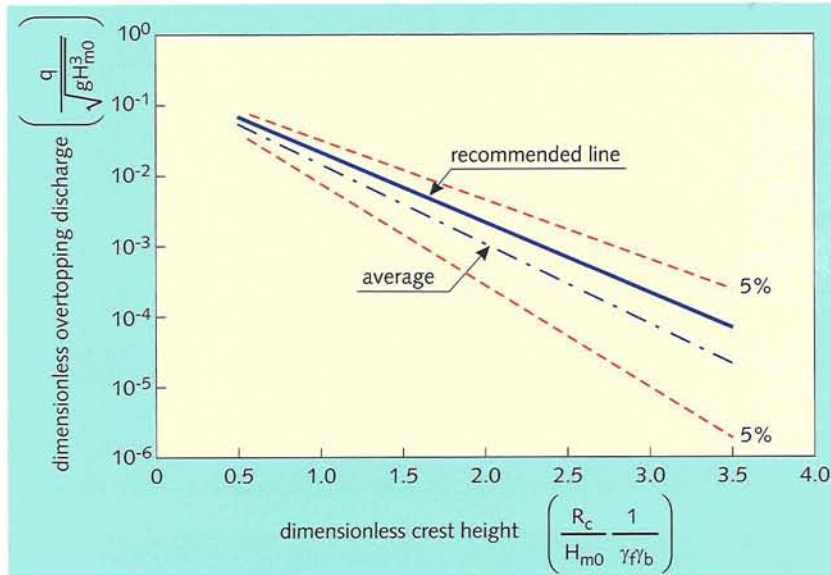


Figure 20:
maximum wave overtopping
achieved with non-breaking
waves

Figure 21 shows an overall view of the measured points related to breaking waves. In this figure the important parameters are given along the two axes, all existing measured points are shown with a mean and 5% lower and upper exceedance limits, and along the vertical axis the application area is also given.

The average of all observations in figures 21 and 22 can be described as:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} \cdot \gamma_b \cdot \xi_0 \cdot \exp \left(-4.75 \frac{R_c}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_t \cdot \gamma_\beta \cdot \gamma_v} \right) \quad (24)$$

(figure 21)

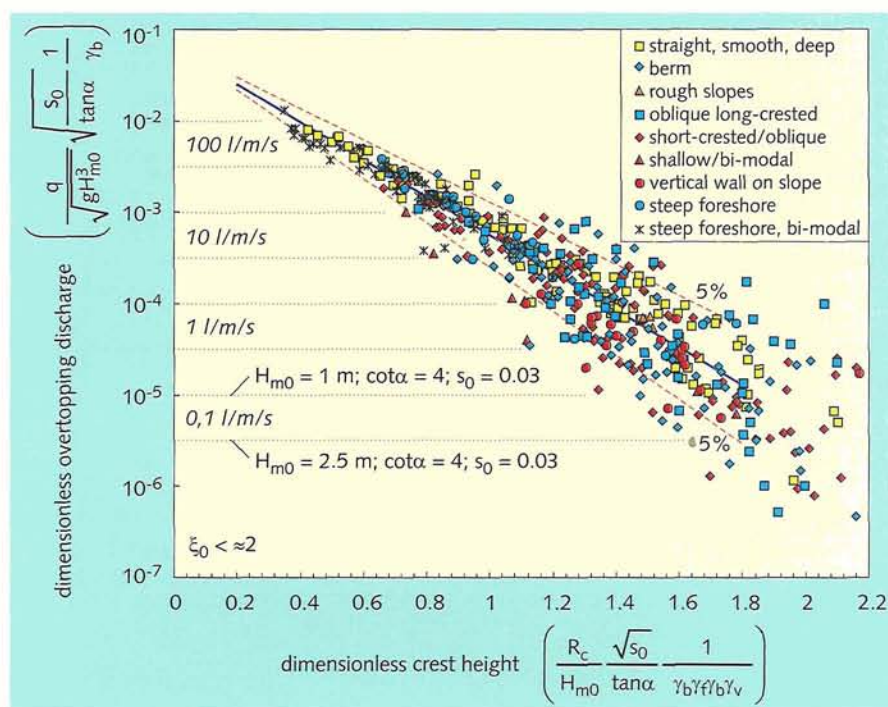
$$\text{with maximum: } \frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp \left(-2.6 \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_t \cdot \gamma_\beta} \right) \quad (25)$$

(figure 22)

The reliability of formula 24 is given by taking the coefficient 4.75 as a normally distributed stochastic function with a mean of 4.75 and a standard deviation $\sigma = 0.5$. Using this standard deviation, the exceedance limits ($\mu \pm x\sigma$) can also be drawn for x plus a number of standard deviations (1.64 for the 5% exceedance limits and 1.96 for the 2.5% under and upper exceedance limits).

Figures 21 and 22 also show some wave overtopping discharges 0.1, 1, 10 and 100 l/s per m, together with an interval for each discharge. The discharges apply for a 1:4 slope and a wave steepness of $s_0 = 0.03$. The uppermost line of the interval applies for a significant wave height of 1.0 m (for, e.g., river dikes) and the lowest line for a wave height of 2.5 m (for, e.g., sea dikes).

Figure 21:
wave overtopping data with
mean and 5% under and
upper exceedance limits and
indication of application area;
breaking waves



The available measured points for the maximum with non-breaking waves are plotted in figure 22. The dimensionless wave overtopping discharge is now given on the vertical axis as:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}}$$

and the dimensionless crest height as:

$$\frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta}$$

3. Wave overtopping

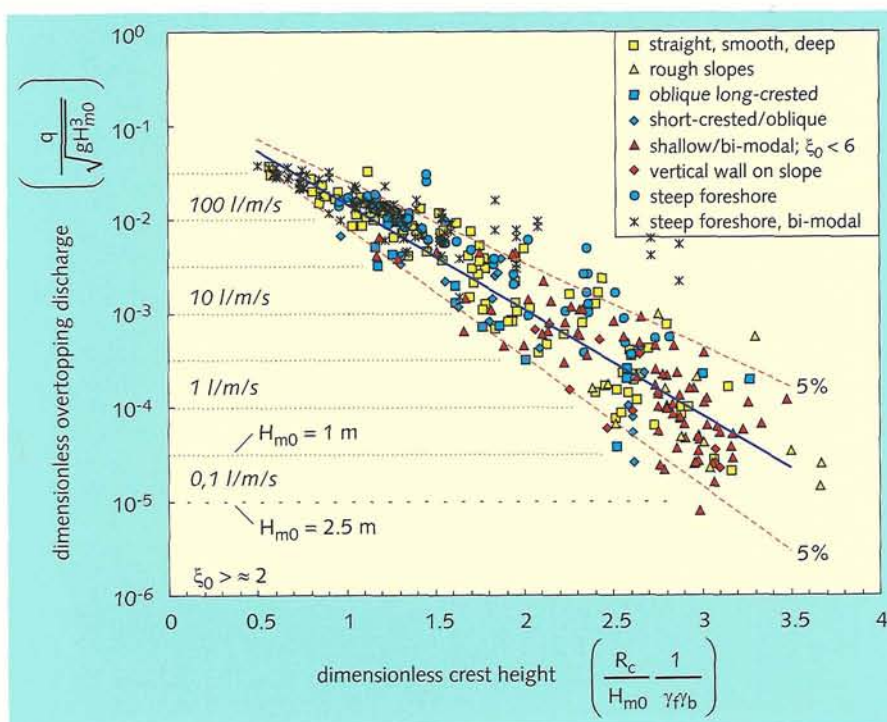


Figure 22:
wave overtopping data with
mean and 5% under and
upper exceedance limits, and
indication of application area;
non-breaking waves

The reliability of formula 25 can be given by taking the coefficient 2.6 as a normally distributed stochastic function with a standard deviation $\sigma = 0.35$. Using this standard deviation, the 5% under and upper exceedance limits are drawn in figure 22. Wave overtopping discharges of 0.1, 1, 10 and 100 l/m per s are also shown on the vertical axis in figure 22. The intervals given apply to a wave height of $H_{m0} = 1$ m (uppermost line) and 2.5 m (lowest line) and are independent of the slope and wave steepness.

As with wave run-up, for deterministic use in practice a slightly more conservative formula should be used than for the average. The two recommended formulae for wave overtopping are formulae 22 and 23, that lie about one standard deviation higher than the average from formulae 24 and 25 (compare also figures 19 and 20). For probabilistic calculations, one can use the given estimates of the average (formulae 24 and 25) and the given standard deviation.

3.2 Influence of shallow or very shallow foreshores

For the wave run-up formulae in Chapter 2 the influence of shallow and very shallow foreshores was directly included in the formulation, see figure 5. The study in this area provided still too little data for also adjusting the formulations for wave overtopping. In the case of very heavy breaking on a shallow foreshore, a spectrum can be 'flattened out' and long waves can be present. A separate formula for calculating wave overtopping is available for this case and this formula must be used because the formulae discussed in section 3.1 can provide a large and sometimes very large underestimate of the wave overtopping.

The effect of shallow or very shallow foreshores is that by relatively gently sloping slopes, milder than 1:2.5, large values of the breaker parameter ξ_0 were found. It is therefore logical to search for a transition to another wave overtopping formula for larger values of ξ_0 .

It is possible that a larger value of the breaker parameter will be found if a very steep slope (1:2 or steeper) is present, with a relatively deep foreshore. In that case the formulae from section 3.1 should be used.

The transition to shallow or very shallow foreshores, for which wave overtopping will be greater than with the formulae from section 3.1, lies at about $\xi_0 = 6$. In order to maintain continuity, the formulae in section 3.1 are used for $\xi_0 < 5$ and the formula is valid for shallow and very shallow foreshores for $\xi_0 > 7$. In the area in between, the logarithm of q is linearly interpolated between $5 < \xi_0 < 7$. The wave overtopping formula for shallow and very shallow foreshores for $\xi_0 > 7$ is:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.21 \cdot \exp \left(- \frac{R_c}{\gamma_f \cdot \gamma_\beta \cdot H_{m0} \cdot (0.33 + 0.022 \cdot \xi_0)} \right) \quad (26)$$

Formula 26 must be used for deterministic calculations as there is a safety margin in it compared to the average prediction. For probabilistic use, the mean should be used with a distribution around this mean. The formula for the mean is:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 10^c \cdot \exp \left(- \frac{R_c}{\gamma_f \cdot \gamma_\beta \cdot H_{m0} \cdot (0.33 + 0.022 \cdot \xi_0)} \right) \quad (27)$$

In formula 27 c is a normally distributed stochastic function with a mean of -0.92 (with $10^{-0.92} = 0.12$) and a standard deviation of 0.24 . Figure 23 gives formulae 26 and 27 with 5% under and upper exceedance limits and available measured points [WL, 1999-1; WL, 1999-2].

3. Wave overtopping

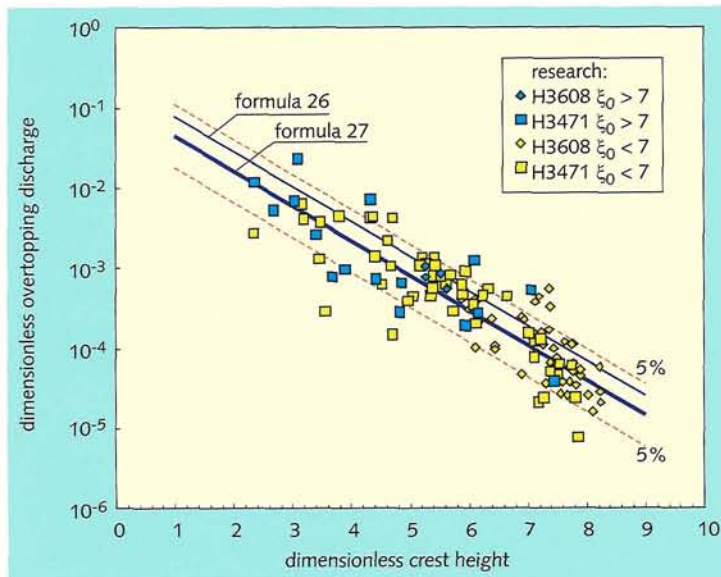


Figure 23:
formulae 26 and 27 for (very) shallow foreshores and 5% under and upper exceedance limits and available measured points

3.3 Interpolations between slopes, berms and foreshores

Procedures are given in section 2.8 for dike profiles that do not conform to the correct definitions of slope, berm and foreshore, and for which wave run-up must be determined using interpolation. For wave overtopping some procedures are required that are slightly different than for those for wave run-up and these procedures are discussed in this section.

Wave overtopping for berm wider than $0.25 L_0$

Wave overtopping for a dike profile with a berm wider than $0.25 L_0$, but less than $1.0 L_0$, is discussed here. This is a slope section that per definition lies between a berm and a foreshore. Concerning wave overtopping, two questions are raised:

- What is the required crest height for a given wave overtopping discharge?
- What is the wave overtopping for a given crest height?

Figure 24 shows a diagram of the procedure:

procedure a:

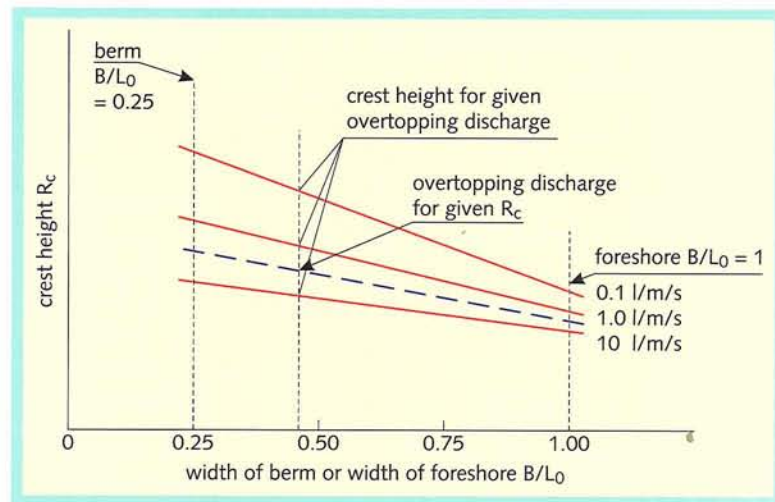
- Determine the required crest height for the given wave overtopping discharge for a berm with a width of $0.25 L_0$ (see also figure 16).
- Determine the required crest height for the given wave overtopping discharge for a foreshore with a length of $1.0 L_0$.
- Interpolate linearly between these two crest heights using B/L_0 as parameter.

procedure b:

- Follow procedure a for a minimum of 2 estimated values for the wave overtopping discharge.
- If the crest height of the actual dike profile does not yet lie between the determined crest heights, then determine some more crest heights such that the point does actually lie between lines.
- Use interpolation to determine the correct wave overtopping discharge.

The answer can also be found using an iterative method.

Figure 24:
determination of wave overtopping for dike profile with gently sloping slope section with length between berm and foreshore



Wave overtopping for a slope between 1:8 and 1:15

The procedure for a slope section that lies between a gentle slope and a berm is as follows, see also figure 25:

- a. What is the required crest height for a certain wave overtopping discharge?
 - Determine the required crest height for the given wave overtopping discharge for a 1:8 slope (see also figure 15).
 - Determine the required crest height for the given wave overtopping discharge for a berm with a 1:15 slope.
 - Interpolate linearly between these two crest heights with the actual slope ($\tan\alpha$) as parameter.
- b. What is the wave overtopping for a given crest height?
 - Follow procedure a for a minimum of 2 estimated values for the wave overtopping discharge.
 - If the crest height of the actual dike profile does not yet lie between the determined crest heights, determine some more crest heights so that the point does lie between the lines.
 - Determine by interpolation the correct wave overtopping discharge.

The answer can also be found using an iterative method.

3. Wave overtopping

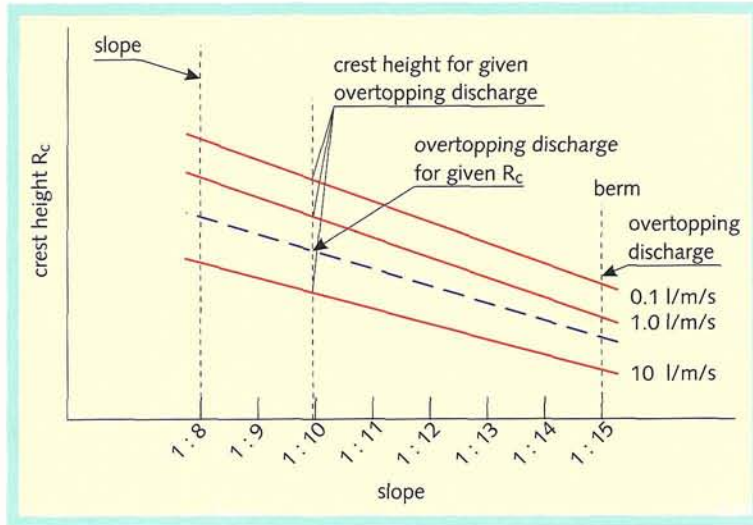


Figure 25:
determination of wave over-
topping for dike profile with
slope section with slope
between 1:8 and 1:15

3.4 Overtopping volumes per wave

The recommended line for the average wave overtopping discharge q is described in section 3.1. The average wave overtopping discharge does not say much about the amount of water that will flow over the crest for a certain overtopping wave. The wave overtopping volumes per wave differ substantially from the average wave overtopping discharge. Using the average wave overtopping discharge the probability distribution function for the wave overtopping volume per wave is calculated. This probability distribution function is a Weibull distribution with a shape factor of 0.75 and a scale factor a , which depends on the average wave overtopping discharge and the probability of overtopping waves. The probability distribution function is given by:

$$P_V = P(\underline{V} \leq V) = 1 - \exp \left[- \left(\frac{V}{a} \right)^{0.75} \right] \quad (28)$$

$$\text{with : } a = 0.84 \cdot T_m \cdot q / P_{ov} \quad (29)$$

where:

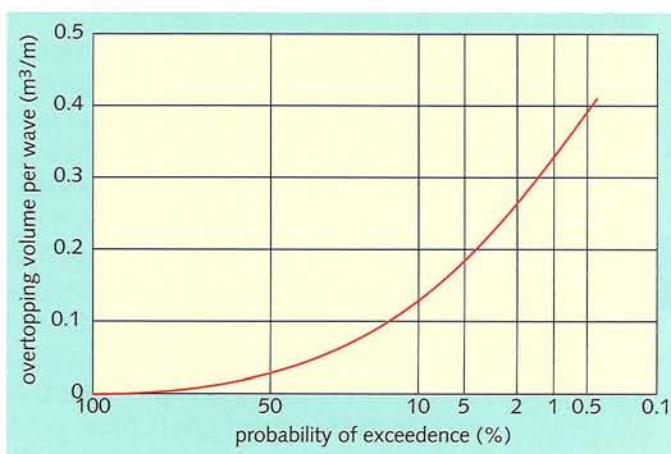
- P_V = probability that wave overtopping volume per wave \underline{V} is greater than or same as V (-)
- V = wave overtopping volume per wave (m^3 per m)
- T_m = average wave period (NT_m is duration of storm or examined time period) (s)
- q = average wave overtopping discharge (m^3/m per s)
- P_{ov} = N_{ov}/N = probability of overtopping per wave (-)
- N_{ov} = number of overtopping waves (-)
- N = number of incoming waves during period of storm (-)

The probability of overtopping per wave can be calculated as follows:

$$P_{ov} = \exp \left[- \left(\sqrt{-\ln 0.02} \frac{R_c}{z^{2\%}} \right)^2 \right] \quad (30)$$

Formula 30 applies to the assumption that the wave run-up distribution conforms to the Rayleigh distribution. The 2% wave run-up can be calculated using formula 3. The influence factors γ_b , γ_f , γ_β , γ_v and the breaker parameter ξ_0 are defined in Chapter 2. To illustrate this figure 26 shows for illustration a probability distribution function based on formulae 28-30. The line shown applies for an average wave overtopping discharge of $q = 1$ l/s per m width, a wave period of $T_m = 5$ s and an wave overtopping probability of $P_{ov} = 0.10$ (10% of the incoming waves).

Figure 26:
probability distribution function for wave overtopping volumes per wave;
 $q = 1$ l/s per m width,
 $T_m = 5$ s and $P_{ov} = 0.10$



This means that $a = 0.042$ (in formula 29) and that the probability distribution function is given by:

$$P_v = P(\underline{V} \leq V) = 1 - \exp \left[- \left(\frac{V}{0.042} \right)^{0.75} \right]$$

The volume for a certain probability of exceeding P_v follows from:

$$V = a \cdot [-\ln(1 - P_v)]^{(4/3)} \quad (31)$$

A first estimation of the predicted value for the maximum volume of one wave that can be expected in a certain period can be gained by filling in the total number of overtopping waves N_{ov} :

$$V_{max} = a \cdot [\ln(N_{ov})]^{(4/3)} \quad (32)$$

In order to give an idea of the relationship between the average wave overtopping discharge q and the predicted value of the maximum volume in the largest wave overtopping wave V_{max} , this relationship is shown for two situations in figure 27. Assumptions are a storm duration of 1 hour, a slope of 1:4 and a wave steepness $s_0 = 0.04$ with a $T_{m-1.0}/T_m$ relationship of 1.15. Relationships are drawn for wave heights of $H_{m0} = 1$ m and 2.5 m.

3. Wave overtopping

For small average wave overtopping discharges, V_{max} is in the order of q times 1000s and for high average wave overtopping discharges in the order of q times 100s.

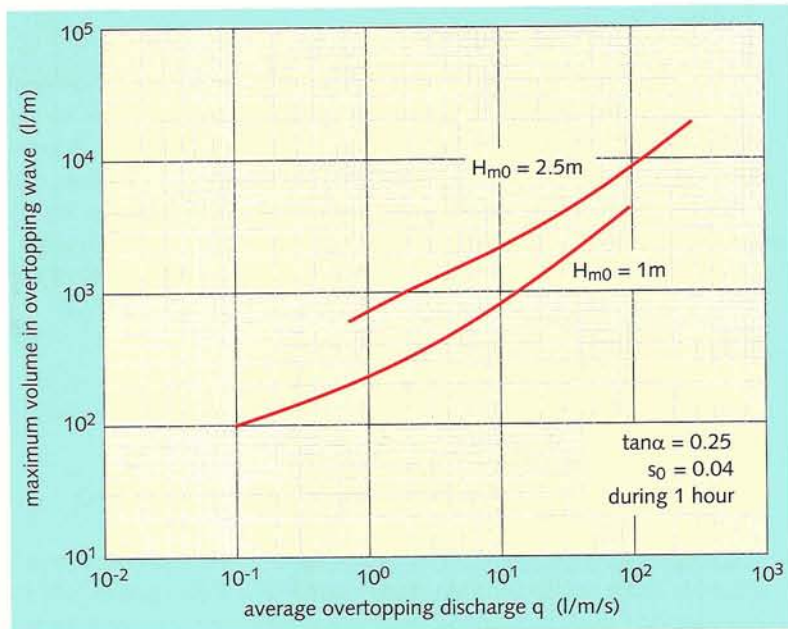


Figure 27:
Relationship between
average wave over-
topping discharge
and maximum
volume of highest
wave overtopping

List of symbols with application area

In the table below, parameters and symbols are shown as used in the report, together with the global application area. For the user, this gives some idea of whether the situation to be calculated is within the area to be applied.

Symbol	Name	Unit	Application area
B	width of berm measured horizontally	m	0-100
D	average diameter of rock armour	m	0.01-1
d_h	berm depth in relation to SWL (negative means berm is above SWL)	m	$-R_c - h_m$
f_b	width of a roughness element (perpendicular to dike axis)	m	0.01-1
f_h	height of a roughness element	m	0.01-1
f_L	centre-to-centre distance between roughness elements, optimum $f_L/f_b = 5-8$	m	0.01-10
g	acceleration due to gravity	m/s ²	9.81
H	wave height	m	0-10
H_{m0}	significant wave height, based on spectrum $\sqrt[4]{m_0}$	m	0-10
$H_{1/3}$	significant wave height, mean of highest 1/3 part	m	0-10
$H_{m0,deep}$	significant wave height in deep water	m	0-10
$H_{m0,toe}$	significant wave height at toe of structure	m	0-10
h	water depth	m	> 0
h_d	dike-table height	m	> 0
h_m	water depth at toe of structure	m	> 0
L_{berm}	horizontal length between two points on slope 1.0 H_{m0} above and 1.0 H_{m0} below middle of berm	m	0-100
L_0	wave length in deep water based on $T_{m-1,0}$	m	0-1000
L_{slope}	horizontal length between two points on the slope $z \geq 2\%$ above and 1.5 H_{m0} below SWL	m	0-100
m_0	surface of energy density spectrum	m ²	0-6
m_{-1}	first negative moment of energy density spectrum	m ² s	0-6
N	number of incoming waves	-	50-50,000
N_{ov}	number of overtopping waves	-	0-20,000
P_V	$P(\underline{V} \geq V)$ probability that overtopping volume \underline{V} is greater than or same as V	-	0-1
P_{ov}	probability of overtopping waves ($P_{ov} = N_{ov}/N$)	-	0-1

Symbol	Name	Unit	Application area
q	average wave overtopping discharge per linear metre of crest	$\text{m}^3/\text{s}/\text{m}$	10^{-6} - 10^{-1}
R_c	crest freeboard in relation to SWL, at position of outer crest line	m	> 0
r_B	influence factor for berm width	-	0-1
r_{dh}	influence factor for berm level	-	0-1
s_0	wave steepness with L_0 based on $T_{m-1.0}$ ($s_0 = H_{m0}/L_0$)	-	0.001-0.07
T	wave period	s	0-25
T_m	average period	s	0-20
$T_{m-1.0}$	spectral wave period = m_{-1}/m_0	s	0-25
T_p	peak period	s	0-25
T_s	significant period	s	0-25
V	volume of overtopping wave per linear metre of crest	m^3/m	0-50
V	coefficient of variation	-	
z	wave run-up height, measured vertically in relation to SWL	m	0-30
$z_{2\%}$	wave run-up height exceeded by 2% of incoming waves	m	0-30
$z_{2\%, \text{berm}}$	wave run-up on a slope with berm	m	0-30
$z_{2\%, \text{smooth}}$	wave run-up on a smooth slope	m	0-30
$z_{2\%, \text{foreshore}}$	wave run-up on a slope with a foreshore	m	0-30
α	angle of average slope	°	0-45
α_{wall}	angle that steep wall makes with horizontal	°	45-90
β	angle of wave attack	°	0 -180
γ_b	influence factor for a berm	-	0.6-1.0
γ_f	influence factor for roughness elements	-	0.5-1.0
$\gamma_{f,i}$	slope section i with a certain influence factor for roughness elements	-	0.5-1.0
γ_v	influence factor for a vertical or very steep wall on a slope	-	0.65-1.0
γ_β	influence factor for angle of wave attack	-	0.7-1.0
ξ_0	breaker parameter based on $T_{m-1.0}$: $\xi_0 = \tan \alpha / \sqrt{s_0}$	-	0.4-20
σ	standard deviation for normal distribution		
μ	mean for normal distribution		

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Appendix 1

Influence factors for the roughness of top layers for wave run-up and wave overtopping

Summary table, based on [DWW, 2002]. The values for the influence factors are based on reference types on which research has been performed, and comparison of photographs of the various slopes.

Code	Description	Influence factor γ_f	Comparison material
1	Asphalt concrete	1.0	Reference type
2	Mastic (asphalt)	1.0	Asphalt
3	Impermeable stone asphalt	1.0	Reference type
4	Open prefabricated stone asphalt mats	0.9	No photograph
5	Open stone asphalt	0.9	Reference type/Fixtone
6	Sand asphalt (temporary or in under layer)	1.0	Reference type
7	Armour rock, impregnated with asphalt (full and saturated)	0.8	Armour rock /asphalt /Vilvoordse stone
8	Brick and concrete, impregnated with asphalt (full and saturated)	1.0	As 7, but smaller roughness
9	Armour rock, impregnated with asphalt (pattern penetration)	0.7	Armour rock /asphalt; single layer 0.8
10	Concrete blocks with angled corners or holes	0.9	Armorflex
11	Concrete blocks without openings	1.0	Reference type
11.1	Haringman blocks	0.9	Reference type
11.2	Diabol blocks	0.8	1/4 blocks higher thus rougher
12	Open block mats, filled with granular material	0.9	Armorflex
13	Block mats without openings	0.95	Closed concrete blocks
14	Concrete sheets of cement concrete or closed colloidal concrete, (poured in situ)	1.0	Closed concrete blocks
15	Colloidal concrete, (open structure)	1.0	Asphalt, barely permeable
16	Concrete sheets, (prefab)	1.0	Closed concrete blocks
17	Growth enabling stone, concrete	0.95	Stone itself somewhat rough, but grass makes it smoother
18	Armour rock, impregnated with cement concrete or colloidal concrete, (full and saturated)	0.8	Armour rock /asphalt /Vilvoordse stone
19	Armour rock, with pattern penetration of cement concrete or colloidal concrete	0.7	Armour rock /asphalt; single layer 0.8
20	Grass, sown	1.0	Reference type
21	Grass, sods or sown, in artificial mats	1.0	Grass
22	Rubble or coarse gravel and other granular materials	0.8	Smaller than quarry stone, less rough, Condition: stable
23	Coarse granular materials - quarry stone packed in metal gauze	0.7	Smaller than armour rock, and permeable
24	Fine granular materials - sand/gravel packed in geotextile	0.9	Smooth, but some permeability
25	Armour rock, (rubble mound)	0.55	Reference type, Single layer 0.7
26	Basalt, set	0.9	Reference type
26.01	Basalt, set, impregnated with poured asphalt	0.95	Basalt, impermeable
26.02	Basalt, set, impregnated with colloidal concrete or cement concrete	0.95	Basalt, impermeable
27	Concrete piles and other non-rectangular blocks		
27.1	Basalt on	0.9	Basalt
27.2	PIT Polygon piles	0.9	Basalt

Code	Description	Influence factor γ_f	Comparison material
27.3	Hydroblock	0.9	Basalt
27.01	Concrete piles or other non-rectangular blocks, impregnated with poured asphalt	1.0	Asphalt, almost smooth and impermeable
27.11	Basalton, impregnated with poured asphalt	1.0	Asphalt, almost smooth and impermeable
27.21	PIT Polygon piles, impregnated with poured asphalt	1.0	Asphalt, almost smooth and impermeable
27.31	Hydroblock, impregnated with poured asphalt	1.0	Asphalt, almost smooth and impermeable
27.02	Concrete piles or other non-rectangular blocks, impregnated with concrete	1.0	Asphalt, almost smooth and impermeable
27.12	Basalton, impregnated with concrete	1.0	Asphalt, almost smooth and impermeable
28	Natural stone, set (Noordse or Drentse stone)	0.75	Single layer of armour rock, somewhat less permeable
28.1	Vilvoordse	0.85	Reference type
28.2	Lessinische	0.85	Vilvoordse, somewhat less rough
28.3	Doornikse	0.9	Basalt
28.4	Small granite	0.9	Basalt
28.5	Granite	0.95	Basalt, somewhat less open
28.01	Natural stone, set, and impregnated with poured asphalt	0.85	Some layers of quarry stone, much less permeable
28.11	Vilvoordse, impregnated with poured asphalt	0.95	Asphalt, almost smooth and impermeable
28.21	Lessinische, impregnated with poured asphalt	1.0	Asphalt
28.31	Doornikse, impregnated with poured asphalt	1.0	Asphalt
28.41	Small granite, impregnated with poured asphalt	1.0	Asphalt
28.51	Granite, impregnated with poured asphalt	1.0	Asphalt
28.02	Natural stone, set, and impregnated with concrete	0.85	Some layers of quarry stone, much less permeable
28.12	Vilvoordse, impregnated with concrete	0.95	Asphalt, almost smooth and impermeable
28.22	Lessinische, impregnated with concrete	1.0	Asphalt
28.32	Doornikse, impregnated with concrete	1.0	Asphalt
28.42	Petit graniet, impregnated with concrete	1.0	Asphalt
28.52	Graniet, impregnated with concrete	1.0	Asphalt
29	Koperslak blocks	1.0	Closed concrete blocks
30	Clay under sand		not applicable
31	Natural rubble mound of rock	0.55	Double layer armour rock; e.g, crumpling berm
32	Clinkers, concrete or brick	1.0	Closed concrete blocks filled in with grass
33	Sand		not applicable

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The Technical Advisory Committee on Flood Defence (TAW) was installed by the Ministry of Transport, Public Works and Water Management in 1965. The Committee advises the Minister on all technical and scientific aspects that might be significant for an efficient construction and maintenance of flood defences, and also on the safety of the areas protected by water defences.

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